ACI News Letter

Vol. 23 No. 6 JOURNAL of the AMERICAN CONCRETE INSTITUTE February 1952

General view of Imhoff tanks at the Sewage Disposal Plant, Indiantown Gap, Pa., after extensive Prepakt repair. The deteriorated areas of all these walls were chipped off, then rebuilt to the original contour with Prepakt Concrete.

Workmen chipping away disintegrated material from back wall of the drying beds.

PREPAKT REPAIRS A SEWAGE DISPOSAL PLANT

The severely deteriorated tank wall and beam structures in this Pennsylvania sewage disposal plant were completely restored by Intrusion-Prepakt methods and materials.

As shown in the photos, the old concrete in these tanks was cleaned away and then rebuilt with high-density Prepakt Concrete while the plant was in partial operation. The rebuilt beam and wall structures now have exceptional resistance to freezing and thawing and chemical deteriorating factors. The exceptionally high bonding strength between Prepakt Concrete and the old concrete provided a true monolithic section, and restored the original strength to all structures.

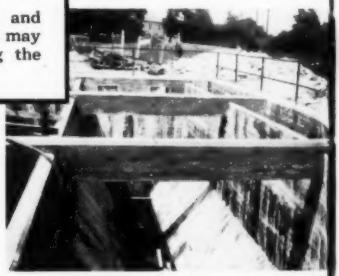
This is another example where Prepakt has been specified to gain those superior Prepakt advantages that contribute to long, maintenance-free life of concrete structures.

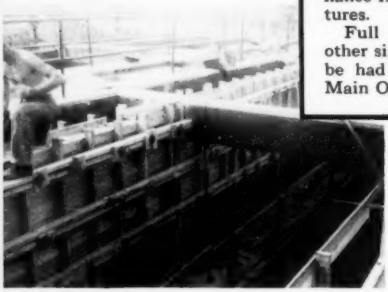
Full information on this and other similar Prepakt projects may be had by calling or writing the Main Office, Cleveland, Ohio.

View showing typical condition of wall before removing old concrete.

View showing extent of disintegration of a beam across Imhoff Tank.

New Outlet Basins built with Prepakt Concrete at the end of Imhoff tanks. Note top of tank wall at left, prior to stripping forms.





Method of forming for rebuilding 6" partition walls. Note Prepakt-rebuilt beam across tank.

New beams and structure on Imhoff tanks, replacing fractured and disintegrated units.

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ACI

News Letter Contents

February 1952

On the Cover—Harvey Whipple, ACI Secretary-Treasurer, who has served continuously for 32 years. He is being honored by a special luncheon Feb. 28, 1952 in conjunction with the 48th annual convention.

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Time Table for 48th Annual Convention

February 26-28, 1952

Netherland Plaza Hotel

Cincinnati, Ohio

Tuesday, February 26

8:30 a.m.—Technical committee meetings

9:00 a.m.—Registration begins

10:30 a.m.—First general convention session—Prestressed concrete—Pavillon Caprice

2:00 p.m.—Convention session—Durability and curing —Pavillon Caprice

8:00 p.m.—Technical committee meetings

Wednesday, February 27

(Concurrent sessions will be held morning and afternoon in the Pavillon Caprice and Parlors A-D)

9:00 a.m.—Convention session—Ultimate load design—Pavillon Caprice

—Convention session—Materials and properties—Parlors A-D

12:15 p.m.—General Luncheon—Hall of Mirrors

2:30 p.m.—Convention session—Design—Pavillon Caprice

—Convention session—Construction—Parlors A-D

7:30 p.m.—Technical committee meetings

Thursday, February 28

9:00 a.m.—Open meeting of ACI Committee 115, Research—Pavillon Caprice

12:15 p.m.—Harvey Whipple Luncheon

2:30 p.m.—Final convention session—Panel discussion —Pavillon Caprice

Convention Program Highlights

In three days of information-packed sessions at Cincinnati those attending the 48th annual convention will find much of interest on concrete and its constituent materials. Session topics include design and construction of prestressed concrete structures, curing and durability of concrete, theoretical and practical considerations in ultimate load design, materials and properties, design of several types of concrete structures, and construction techniques. Synopses of the convention papers were published in the January *News Letter*.

An annual feature, the open session of Committee 115, Research, will summarize, in the usual brief fashion, current research projects relating to concrete. The panel discussion, with an entirely new cast of concrete experts, will emphasize the problems on the small concrete job.

Prestressed concrete

The relative economy of conventional and prestressed reinforced concrete reservoirs will be discussed by G. C. Ernst, C. O. Brunken and A. R. Riveland, University of Nebraska, Lincoln, Neb. A feature of their study is the number of different designs considered; 260 complete reservoir and wall designs being analyzed. From this mass of data the authors find that prestressed concrete reservoirs have a lower first cost than conventional concrete if the unit cost of the prestressed wall in place does not exceed approximately twice that for the conventional wall. Above ground reservoirs are found to be more economical than underground reservoirs in capacities less than 2.5 million gallons.

In many past papers on prestressed concrete emphasis has been on theoretical design considerations. Clair L. Johnson, Johnson and Anderson, Pontiac, Michigan, however, deals with the problems in translating the design into the finished structure. He tells of difficulties with a multiplicity of special shape blocks which greatly complicated their manufacture on standard concrete block machines. Revisions in design eliminated this difficulty and changes in erection technique reduced block breakage and made grouting easier.

Curing

Since curing of concrete has an important bearing on its strength and durability a round-table discussion of this phase of concrete construction is sponsored by Committee 612, Recommended Practice for Curing Concrete, Mark Morris, chairman.

As a prelude to other papers to follow, A. G. Timms, National Bureau of Standards, Washington, D. C., will review present thinking on the fundamental principles of curing. He will consider the various general means of controlling moisture and temperature in new concrete and the effect of these factors on the characteristics of the cured concrete. Curing methods fall into two general classifications; those that supply water throughout the early hydration process and tend to maintain a uniform temperature by ponding, sprinkling, application of wet burlap or cotton mats, wet earth, sawdust, hay or straw; and those designed to prevent loss of water but have little influence in maintaining a uniform temperature. The latter include the application of waterproof paper or impermeable membranes.

D. L. Robinson, Bureau of Materials, Missouri State Highway Dept., Jefferson City, Mo., will report on methods of curing permitted under current state highway department specifications. Saturated coverings of cotton felt or jute mats or two thicknesses of burlap are almost universally permitted. Other types discussed are covering with waterproof paper, saturated, earth or ponding, sealing with membranes and covering with hay or straw.

Factors in curing structural concrete will be summarized by H. J. Gilkey, Iowa State College, Ames, Iowa. Where and how these differ from those for other types of concrete

Continued on p. 6

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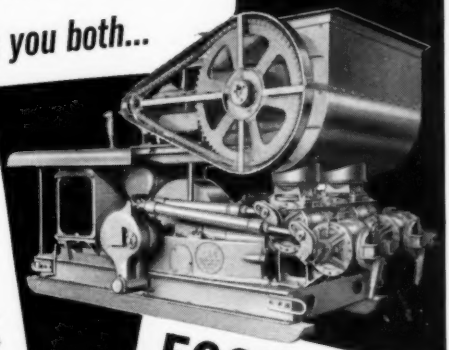

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Program highlights

Continued from p. 4

will be pointed out. Current practices, especially form removal and temperature and moisture control in cold weather, will be discussed.

The specialized requirements for curing mass concrete will be presented by W. R. Johnson, Corps of Engineers, Portland, Ore., and Roy W. Carlson, consulting engineer, Berkeley, Calif. Water and membrane curing will be discussed and data given on protection afforded by tarpaulins and wood and steel forms to concrete placed at low temperatures.

The use of white-pigmented liquid sealing compounds in the curing of canal linings will be reported by G. E. Burnett, Bureau of Reclamation, Denver, Colo. Good material, proper time of application, adequate film thickness and uniform coverage are important factors in this type of curing.

In closing the round-table discussion of curing, H. C. Vollmer, Emergency Transportation Dept., Washington, D. C., will ist and describe available curing agents and give their advantages and disadvantages and reasons for their increased use under various circumstances.

Durability

A group of papers on durability of concrete will report on problems peculiar to pavements and railroad and hydraulic structures.

Frank H. Jackson, Bureau of Public Roads, Washington, D. C., will define structural performance and durability as applied to concrete pavements. The role of air entrainment and the effect of other factors on the durability of pavements, particularly in the northern states, will be emphasized.

Two papers will report on the durability of concrete in hydraulic structures. L. H. Tuthill, Bureau of Reclamation, Denver, Colo., will discuss the necessity for controlling volume change and providing impermeability in hydraulic works. Mix design, placement methods, use of admixtures and other measures to increase serviceability will be considered.

Byram W. Steele, consulting engineer, Miami, Fla., for many years with the Corps of Engineers, will propose measures for

increasing the impermeability of hydraulic structures and thus increasing their service life.

C. B. Porter, Chesapeake and Ohio Railway, Richmond, Va., will summarize results of a survey of the general condition of concrete railroad structures and report on the progress of a research program on surface waterproofers.

Further information on field studies of durability of railroad concrete will be supplied by R. W. Gilmore, Baltimore and Ohio Railroad, Cincinnati, Ohio. Design modifications and changes in construction and maintenance procedures to prolong the useful life of railroad structures will be reviewed.

Ultimate load design

This important session on the background and application of ultimate design methods will be introduced by Leo H. Corning, Portland Cement Assn., who will be followed by Boyd G. Anderson, Ammann and Whitney, New York, N. Y. Mr. Anderson will discuss the reasons for this design procedure.

Eivind Hognestad, University of Illinois, Urbana, Ill., will trace the development of design methods for reinforced concrete structural members through the three stages of an engineering science and discuss various ultimate design equations in relation to the three prevailing philosophies of failure.

C. P. Siess will describe tests to determine the manner of failure and ultimate strength of reinforced concrete structural members. Emphasis will be on tests in the United States, but foreign tests will also be mentioned. Available test data will be reviewed and evaluated and the additional data needed will be considered.

The practical application of ultimate load theories to the design of sections will be demonstrated by Raymond C. Reese, consulting engineer, Toledo, Ohio. Designs will be developed for slabs and beams and for columns with and without bending by the conventional and ultimate load design methods. T. Y. Lin, University of California, Berkeley, California, will compare load factors in ultimate design and allowable stresses in conventional design. The basis for the determination of load factors will be considered and tentative recommendations for their selection will be made.

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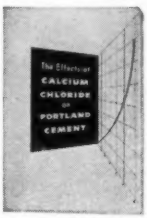

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Materials and properties

Studies made at the Waterways Experiment Station, Jackson, Miss., on thermal expansion of aggregates and its relation to concrete durability will be summarized by Edwin J. Callan. Methods used for the rapid, simple determination of the thermal coefficients of fine and coarse aggregate will be described and the application of these data to the development of durability factors for concretes indicated.

A study of the effect of temperature changes on concrete as influenced by aggregates will be reported by Delmar Bloem, Stanton Walker and W. G. Mullen, National Sand and Gravel Assn., Washington, D. C. A number of conclusions on the relationships developed will be presented.

Although calcium chloride has been used for many years to accelerate early strength of concrete its effect on other properties are not as well known. J. J. Shideler, Bureau of Reclamation, Denver, Colo., will present test results on the effect of type of cement, age, temperature and mix proportions on the strength of concrete as well as the effect of calcium chloride on volume change, heat generation, time of set, alkali-aggregate reaction, resistance to abrasion and sulfate attack of concrete.

C. H. Scholer and G. M. Smith, Kansas State College, Manhattan, Kan., will describe laboratory tests of beam specimens in which 20 to 30 percent of the cement was replaced by fly ash. Fly ash effectively inhibited cement-aggregate reaction in the concrete while improving workability and finishing.

Design

Field practice in lightweight concrete will be discussed by J. A. Murlin of George L. Dahl, Architects and Engineers, and Cedric Willson, Texas Industries, Dallas, Texas. Concrete made with expanded shale and clay aggregates will be compared with that made with other lightweight materials and the usual heavier materials. The design of lightweight structural members and their economy in comparison with heavy concrete will be considered with due regard to the cost differential between the two materials.

For economical design of concrete beams Herbert A. Sawyer, Jr., University of Connecticut, Storrs, Conn., will present charts and relationships from which cross-sectional dimensions for maximum economy of materials of a beam or slab may be determined for a given span and loading.

James P. Michalos, Iowa State College, Ames, Iowa, will give a numerical procedure for the analysis of single-span skewed rigid frames and arches subjected to loads or deformations in any direction.

Arthur R. Anderson, consulting engineer, Tacoma, Wash., will describe tests of slabs reinforced with wire mesh and give data on the bond developed between the wire and the slab concrete as well as the ultimate strength developed in the mesh itself.

Construction

Applications of vacuum concrete to a number of types of structure in several European and North African countries will be shown by K. P. Billner, Vacuum Concrete Corp., Philadelphia, Pa.

Proper use of concrete in small building construction will be discussed by C. O. Christenson, National Assn. of Home Builders, Washington, D. C. Emphasis will be on the application of lightweight aggregates, manufacture and use of prefabricated building elements, and the control of concrete quality in small building work where the services of a concrete expert usually are not available.

L. H. Tuthill, R. E. Glover, C. H. Spencer and W. B. Bierce, Bureau of Reclamation, Denver, Colo., will tell about their studies of insulation for protection of new concrete in winter. Insulation of forms eliminated special enclosures, fuel costs, and the danger of fires with a resultant reduction in cost.

A sound movie produced by the Centriline Corp., New York, N. Y., will show the preparation and lining of pipes with mortar by a machine process.

Research session

The annual open session of Committee 115, Research, will feature the usual brief reports of interesting research projects now in progress. Described will be preliminary results on fatigue tests of concrete beams, weathering and freezing and thawing tests, impulsive loading, stress distribution, and volumetric changes.

Continued on p. 10

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Program highlights

Continued from p. 8

Panel discussion

Past president H. J. Gilkey will guide a panel of experts in a discussion of problems relating to concrete work. Emphasis this year will be on questions dealing with the small concrete job.

Bylaws changes

A number of changes in the Bylaws of the Institute will be presented for consideration by the convention. The composition and activities of the honorary membership committee are spelled out in greater detail with an added requirement that the committee submit an annual report to the Board of Direction each year, whether or not honorary members are proposed. Under the same article the various classes of members and their privileges are clarified.

Section 1 of the article pertaining to officers of the Institute sets forth the manner in which they attain their positions. Provisions for the appointment of the Secretary-Treasurer place him in office at the pleasure of the Board of Direction rather than on an annual basis as provided in the present Bylaws. The composition of the proposed Board of Direction puts the Secretary-Treasurer in the role of an advisor rather than his present position as a member of the Board.

Further proposals are editorial changes in the provisions for additional nomination of officers and members of the committee on nominations, vacancies in offices, and the removal of the Secretary-Treasurer from the Executive Committee.

Editorial changes are proposed in articles specifying meetings of the Board of Direction and the date for canvassing letter ballots.

EXPERTS FOR CONVENTION PANEL DISCUSSION

By H. J. GILKEY*

The following men are scheduled to serve as the "Board of Experts" for the convention panel session, Thursday afternoon, February 28:

M. D. CATTON
Portland Cement Assn., Chicago, Ill.

R. E. COPELAND
National Concrete Masonry Assn., Chicago, Ill.

R. W. GILMORE
Baltimore and Ohio R. R., Cincinnati, Ohio

EDWARD C. HARDING
Edward C. Harding Co., Cincinnati, Ohio

EIVIND HOGNESTAD
University of Illinois, Urbana, Ill.

CHARLES T. KENNEDY
Ferro Concrete Construction Co., Cincinnati, Ohio

F. N. MENEFEE
University of Michigan, Ann Arbor, Mich.

NOLAN D. MITCHELL
National Bureau of Standards, Washington, D. C.

J. A. MURLIN
Geo. L. Dahl, Architects and Engineers, Dallas, Tex.

JOHN P. THOMPSON
Portland Cement Assn., Chicago, Ill.

C. E. WUERPEL
Marquette Cement Manufacturing Co., Chicago, Ill.

The final list was arrived at through an interesting series of operations involving elimination, selection and circumstance.

From an initial circularization of members of the Board of Direction and others for suggestions, there was assembled a sizeable compilation representing a wide range of concrete specialties, professional affiliation and geographical distribution.

Eliminated immediately was every man who had served as a member of either of the two preceding convention panel groups, after

Continued on p. 29

*Chairman, panel session.

The Centennial of Engineering, 1952

The Centennial of Engineering, in which 48 national and international engineering societies will participate with special ceremonies, will mark the 100th birthday of the formal establishment of engineering in this country as distinct from military engineering on a recognized professional career basis. It was in 1852 that the need of an organization where engineers might share their ideas and the results of their work was realized in the founding of the American Society of Civil Engineers. The term "civil" had another connotation than that of today. It was used to differentiate the work of the civilian engineer from that of the military engineer.

The Centennial celebration is planned in three major parts:

1) The greatest convocation of engineers ever assembled with 48 or more technical societies with a combined membership of over 300,000 meeting jointly in Chicago between September 3 and September 15, 1952.

2) A dynamic exhibit at the Museum of Science and Industry, to remain for five years or more, epitomizing all engineering progress and its effects on our way of life. More than 8 million people are expected to visit the exhibit.

3) A dramatic presentation to tell the story to the general public. It will run through the summer months of 1952 and will portray factually and with real equipment civilization's march forward, particularly in this last century of engineered progress.

The main purposes of the Centennial have been summed up by Lennox R. Lohr, president, Centennial of Engineering, 1952, Inc., as:

1) Make known the contributions of our engineers in peace and war to our national progress.

2) To personalize the engineer in his true role.

3) To stimulate young men to study engineering to fill the future's great need for technically trained men in research and industry.

4) To depict the role of industry with its mass production as essential to our high standard of living and its dependence on engineering and on management to maintain and increase our prosperity.

ACI participation planned early

The American Concrete Institute was one of the first of the many invited societies to list itself among the participants. At its fall meeting in October 1950 the Board agreed to participate and at its annual meeting in San Francisco in 1951, committed itself definitely to hold the 1952 regional meeting at Chicago.

ACI Directors H. C. Delzell and C. E. Wuerpel have been named chairmen of ACI's general arrangements and technical program committees, respectively. By virtue of these responsibilities they are members of the Centennial of Engineering General Arrangements Co-ordinating Committee and Technical Program Co-ordinating Committee.

Evidence that ACI's part in the convocation will be substantial and well planned will be found in the following letter received from the Technical Program Committee.

"Dear Mr. Whipple:

There follows the tentative technical program which has been developed by this committee for joint sponsorship by the ACI and the ASCE for sessions to be held on September 11th and 12th, 1952, during the Centennial of Engineering:

'September 10th—Registration at the Hotel Conrad Hilton (the Stevens) and trips to the Museum of Science and Industry to view the extremely interesting exhibit epitomizing all engineering progress in the past century and its effects on our way of life. Also, the International Engineering Banquet will be held that night, at which the principal speaker is expected to be Herbert Hoover, and at which other outstanding personalities associated with engineering progress will speak.

'September 11th—Joint Session of ACI and ASCE on *Prestressed Concrete.* The session is to be in two parts, from 9:30 a.m. to 12:00 noon and from 2:00 to 4:30 p.m.

A. E. Cummings, Director of Research, Raymond Concrete Pile Company, New York, Chairman of the Joint Committee of ACI and ASCE on Prestressed Concrete, has consented to be Chairman of this all-day session.

'Professor M. J. Holley, Jr., of Massachusetts Institute of Technology has consented to open the program with a paper discussing prestressed concrete in general; its development in Europe and in this country; and generally to bridge the gap between the First U. S. Conference on Prestressed Concrete held during September, 1951, at M.I.T.

'Tentative arrangements have been completed for about seven additional papers on various aspects of the design, construction, and use of prestressed concrete in the United States.

'September 12th—Joint Session of the ACI and the ASCE on *Precast and Thin Shell Concrete Design and Construction.* Approximately eight papers averaging about 25 minutes each with provision for "planted" active, but limited, discussion to break the monotony of presentation. Sessions from 9:30 to 12:00 noon and from 2:00 to 4:30 p.m. Mr. Stewart Mitchell, Principal Bridge Engineer, California Highway Department, a member of ACI, and new Chairman of the Structural Division of ASCE, has accepted our invitation to serve as Chairman.

'There is no room for doubt that the Centennial of Engineering, the Convocation of Engineers, during the period September 3-13, 1952, and the ACI regional meeting on September 10, 11, and 12 will be affairs of unrivaled interest to all members of this Institute and to the engineering and constructing fraternity."

Yours very truly,

Technical Program Committee

C. E. Wuerpel, Chairman
A. E. Cummings
L. H. Corning
Prof. M. J. Holley, Jr.
H. F. Thomson, ex officio

Headquarters at the Conrad Hilton

The ACI regional meeting headquarters will be the Conrad Hilton Hotel. Other societies with headquarters at the Conrad Hilton will be: American Society of Civil Engineers, American Institute of Consulting Engineers, Engineers Council for Professional Development, the National Council of State Boards of Engineering Examiners, the Society of American Military Engineers.

ACI members desiring to make room reservations in advance for the period of the Convocation, September 3-13, 1952, are urged to write directly to the Conrad Hilton Hotel, Chicago, Ill., noting that the request is in connection with attendance at the ACI regional meeting.

Concrete Industry Board for quality improvement

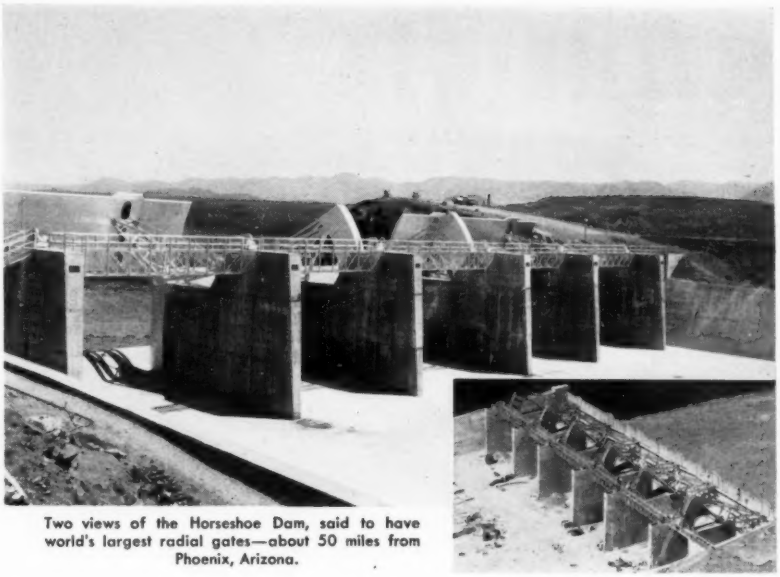
Engineers, architects, material suppliers, inspection firms, contractors and technical societies concerned with concrete construction have set up a Concrete Industry Board in New York City to improve the quality of reinforced concrete construction in the metropolitan area.

The board is sponsored by the Cement League, a component of the Building Trades Employers Assn., which deals mainly with labor-management problems.

Roger H. Corbetta, president of Corbetta Construction Co., is chairman of the board; John F. Hall, district engineer, Portland Cement Assn., is secretary; and Dugald J. Cameron, Fireproof Products Co., is treasurer. Members of the board of directors of the organization are being appointed by ACI, AIA, ASCE, CRSI, PCA, Ready-Mix Concrete Producers, Structural Engineers Society, and testing laboratories.

ASTM Committee Week

More than 50 ASTM committees are planning to meet during ASTM Committee Week, March 3-7, 1952, at the Hotel Statler, Cleveland, Ohio. Some of the committees intending to meet are ASTM Committees C-1, Cement; C-9, Concrete and Concrete Aggregates; and D-4 Road and Paving Materials. A basic purpose of Committee Week is to permit a large number of men who serve on different committees to attend meetings of these frequently diverse groups in a concentrated period.



Two views of the Horseshoe Dam, said to have world's largest radial gates—about 50 miles from Phoenix, Arizona.

IN the construction of the Horseshoe Dam, builders faced the problem of obtaining satisfactory placeability, yet maintaining designed strength, in the comparatively thin sections of the piers which were heavy with steel reinforcement.

The builders' knowledge of the ability of Pozzolith — the cement dispersing agent — to improve plasticity even though water is reduced as much as 15%, assured them that it was the answer.

In addition to providing the most economical solution to the placing problem, Pozzolith produced good cohesiveness, thereby minimizing finishing costs.

Increased bond of concrete to steel and great durability were other benefits obtained with Pozzolith.

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Close fitting pre-cast bascule weights which counterbalance the radial gates, also I-beam seal at bottom of gates, were precision grouted with another Master Builders' cement dispersion product—EMBECO—which produces non-shrink grout of high strength.

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Symposium on Laboratory Design for Handling Radioactive Materials

By NOLAN D. MITCHELL*

A symposium sponsored by AIA and the Atomic Energy Commission was held under the auspices of the Building Research Advisory Board of the National Research Council at the National Academy of Sciences on November 27-28, 1951.

A large proportion of the subject matter related to projects of the Atomic Energy Commission and of contractors to that organization. The extent of interest in the subject is indicated by the report that there are now about 1500 laboratories, hospitals and clinics using radioactive materials. It was also reported that the number of users is growing at an accelerating rate.

Concrete construction was predominant among the examples shown and was also used as the protective screening material wherever space permitted. Those describing recently constructed projects emphasized the desirability of having the building designed to permit the ready removal or relocation of partitions and equipment for changes necessitated by unforeseen developments. Modules of 4 or 8 ft were suggested as being desirable to guide initial layout of the structural work.

Examples of laboratory buildings shown had clear spans of 24 ft with level and continuous ceilings. This allowed placing movable partitions at any desired location. The module for equipment or space could be 24 ft or any fraction thereof.

Those experienced in laboratory work stressed the desirability of liberal space allotment since distance provided excellent protection. The "hot" areas should be kept at a distance from the places where other work is being carried on.

Rapid extension of the use of radioactive materials in many lines of research and industry makes necessary a wide dissemination of information on laboratory techniques and laboratory design if such work is to be done safely. By means of this symposium, the sponsoring agencies and the Building Research Advisory Board have made available to the interested professions a fund of information developed through practice and experience. It is the intention of the Building Research Advisory Board to publish the material presented.

The total registration for the meeting was 232. Approximately one-fourth of the registrants were architects and another fourth represented research laboratories. Eight of the registrants represented three Canadian governmental organizations.

The Current Reviews section of the ACI JOURNAL will contain reviews of the various papers presented at the symposium as they become available.

*ACI representative, consultant, Building Technology Division, National Bureau of Standards, Washington 25, D. C.

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Who's Who This Month

James P. Michalos

James P. Michalos, associate professor of civil engineering, Iowa State College, Ames, Iowa, is the author of "Analysis of Skewed Rigid Frames and Arches," p. 437.

Professor Michalos has had extensive experience with consulting firms, industry and TVA design, analysis and construction of various types of structures. Prior to joining the staff at Iowa State College, he taught at Montana State College and Syracuse University.

Professor Michalos is a civil engineering graduate from the University of Wisconsin and received a master's degree from Northwestern University.

He has been affiliated with ACI since 1949 and is a member of ACI Committee 314, Rigid Frame Bridges.

C. H. Scholer and G. M. Smith

C. H. Scholer, professor of applied mechanics, and G. M. Smith, assistant professor of applied mechanics, Kansas State College, Manhattan, Kans., describe "The Use of Chicago Fly Ash in Reducing Cement-Aggregate Reaction, p. 453.

Professor Scholer has been an ACI member since 1924, and has been active on ACI Committees 115, Research, and 215, Fatigue of Concrete.

After receiving a BS degree in civil engineering from Kansas State College in 1914, he was head chainman, U. S. General Land Office surveys, Santa Fe, N. M., 1914-15; chainman, maintenance of way and realignment problems, Atchison, Topeka and Santa Fe R. R., Topeka, Kans., 1915-16; assistant engineer in charge of surveys and preparation of plans, and supervision of the construction of roads, pavements and bridges for the Extension Division, Kansas State College, Manhattan, Kans., 1916-17; assistant bridge engineer in charge of the design and supervision of construction of highway bridges for the Kansas State Highway Commission, 1917-18; professor and head, Department of Applied Mechanics, since 1919 except for a leave of absence to take charge of investigational work in New York and Pennsylvania for the Portland Cement Assn., February 1939 to September 1940.

Professor Scholer is associated with ASCE, NSPE, ASTM, ASEE, Kansas Engineering Society, the Highway Research Board and has been honored by election to Sigma Xi and Phi Kappa Phi.

Professor Smith for the past three years has been responsible for the execution of the research program in a study of special concrete problems related to the durability of concrete. Part of his time is also given to teaching undergraduate and graduate courses in the field of applied mechanics.

He graduated from Kansas State College with a BS degree in mechanical engineering in 1947, and a master's degree in applied mechanics in the spring of 1951. Professor Smith is a registered engineer in Kansas and a member of ACI.

Henry J. Cowan

"Direct Design of Rectangular Reinforced Concrete Columns with Bending About an Axis of Symmetry," p. 465, is by Henry J. Cowan, department of civil engineering, University of Sheffield, Sheffield, England.

Since 1948 he has been a lecturer in civil engineering in charge of the concrete laboratory at Sheffield, as well as special lecturer in architecture. A graduate of Manchester University, Mr. Cowan received his MS there in 1940. He served with the Royal Engineers until he was wounded in Holland in 1945.

Returning to England, he was assistant engineer with Higgs and Hill, civil engineering contractors of London, and later senior designer in structural and mechanical development with a subsidiary of Associated Light Metal Industries. Prior to joining the University of Sheffield staff he was assistant lecturer at the University of Wales, Cardiff, Wales. He is the author of papers on concrete and reinforced concrete design in several British publications.

Edwin J. Callan

Edwin J. Callan, chief, Thermal Research and Field Durability Section, Concrete Research Division, Waterways Experiment Station, Vicksburg, Miss., contributes "Thermal Expansion of Aggregates and Concrete Durability," p. 485.

Presently, Mr. Callan is engaged in studies of thermal and elastic properties of concretes and components, dynamic testing, weathering tests at both the laboratory and field exposure stations in Maine and Florida, and statistical analyses.

Following graduation from Manhattan College in 1943, he was immediately called on active duty with the Chemical Warfare Service, U. S. Army, and served in the United States and in the Mediterranean and European theaters. After release, he accepted a position as physicist with the Central Concrete Laboratory, Mt. Vernon, N. Y., and was transferred to the Concrete Research Division, Waterways Experiment Station, Jackson, Miss., in 1946.

Mr. Callan has also taken additional courses in mathematics from the University of Chicago and military radiological and electronics courses. Presently, he is active in the U. S. Air Force Reserve and is a member of the American Physical Society, American Chemical Society, American Geophysical Union and Society of American Military Engineers.

George C. Ernst, C. O. Brunken and A. R. Riveland

"Relative Economy of Prestressed and Conventional Reinforced Concrete Reservoirs," p. 505, is presented by George C. Ernst, C. O. Brunken and A. R. Riveland, department of civil engineering, University of Nebraska, Lincoln, Nebr.

Professor Ernst graduated from the University of Michigan in 1925 with a BS in civil engineering, and has been associated with academic work since 1930 at Iowa State College, University of Maryland and University of Nebraska.

Professor Ernst is a registered professional engineer in Iowa. He is affiliated with ASCE, ASTM, SPEE, Sigma Xi, Tau Beta Pi and has been an ACI member since 1935.

Mr. Brunken attended the University of Nebraska and received his BS degree in civil engineering in 1949 and his Master's degree in 1951. Since 1949 he has been teaching in the Department of Civil Engineering, University of Nebraska.

Mr. Brunken currently is participating in experimental work on the testing of thin-shell members. He spent three summers surveying and inspecting bituminous and concrete materials.

Mr. Riveland received a BS in civil engineering from the University of North Dakota in 1945. Following graduation he was employed by Lium and Burdick, Engineers, Grand Forks, N. D., for nearly two and one-half years. His duties entailed writing of specifications and estimating costs for municipal sanitary engineering projects, including concrete water storage reservoirs and water and sewage treatment plants.

Since September 1949, Mr. Riveland has been an instructor in the Department of Civil Engineering, University of Nebraska. In addition to teaching, he is engaged in experimental work concerning thin-walled, reinforced concrete domes.

New engineering program available at University of Michigan

A new program leading to a BS degree in materials engineering is now available in the College of Engineering at the University of Michigan.

The first-year program is identical with the schedule taken by all other engineering students. Specialization begins in the second year and becomes more intensive in later portions of the program.

Materials studied include cements, wood, plywood, metals, rubber, plastics, etc. Emphasis is placed upon the corrosion resistance of these materials and their various protective coatings, such as paints, varnishes, lacquers and enamels.

The new program is similar in many respects to metallurgical engineering except the materials engineer will take more advanced mathematics, stress analysis, machine shop, machine design, specifications, corrosion and high temperature properties, and operations of fabrication.

After graduation, materials engineers are eligible to study toward advanced degrees in chemical, metallurgical and other branches of engineering.

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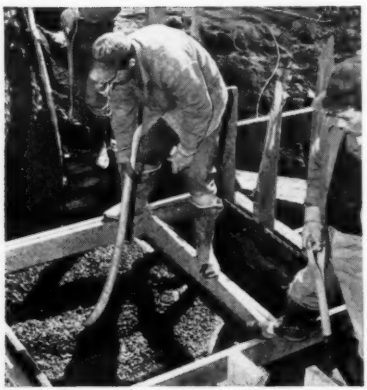

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Positions and Projects — ACI Members

Recent appointments to ACI technical committees

Committee 325, Structural Design of Concrete Pavements for Highways and Airports

Franklin B. Brown
Managing Director
Wire Reinforcement Institute
1049 National Press Bldg.
Washington 4, D. C.

Committee 604, Winter Concreting Methods

M. F. Macnaughton, *chairman*
Vice-President
Milton Hersey Co., Ltd.
980 St. Antoine St.
Montreal 3, Quebec
Canada

Lewis H. Tuthill
Concrete Engineer
Bureau of Reclamation
Denver Federal Center
Denver, Colo.

D. O. Robinson
Chief Engineer, Dept. of Sales
Canada Cement Co. Ltd.
Room 1103, Bank of Montreal Bldg.
50 King St.
Toronto 1, Ont., Canada

Committee 711, Precast Floor Sytsems for Houses

Laurence G. Farrant
Consulting Engineer
908 Langford Bldg.
Miami, Fla.

Committee 716, High Pressure Steam Curing

E. T. Andrews
Vice-President
Pennsylvania Glass Sand Corp.
Hancock, W. Va.

ASTM Committee C-1, Cement

Stanton Walker, *ACI representative*
Director of Engineering
National Sand and Gravel Assn.
1325 E St., N.W.
Washington 4, D. C.

Clayton appointed to post in India

Kenneth W. Clayton, who left India two and a half years ago for further architectural studies in Australia, has obtained his Bachelor Degree of Architecture with the Associateship of the Royal Institute of British and Australian Architects and has just been appointed to the post of Assistant Professor of Architecture at the Bengel Engineering College, Calcutta, India.

Rusk elected association president

Herbert Rusk, president and general manager of Ready-Mixed Concrete, Inc., Mansfield, Ohio, was elected president of the Ohio Ready Mixed Concrete Assn. at its annual convention in Cincinnati.

Loveland named works manager

Russ A. Loveland has resigned as superintendent of Plant No. 4, Pennsylvania-Dixie Cement Corp., Nazareth, Pa., to become works manager for Halliburton Portland Cement Co., Corpus Christi, Texas. Mr. Loveland has had more than 30 years' experience in the cement industry, starting in 1920 as mix chemist at the Hanover, Mont., plant of the Three Forks Portland Cement Co.

Vlad appointed research engineer

Ernest J. Vlad, partner in Frank J. Kornacker and Associates, has been appointed research engineer in the structural department at Armour Research Foundation of the Illinois Institute of Technology.

Mr. Vlad has specialized in structural design and has had experience in ordnance. A graduate of the Illinois Institute of Technology in 1947, he has been associated with Kornacker and Associates since that time.

He is a member of ASCE and has been affiliated with ACI since 1947.

S. L. Meyers

S. L. Meyers, formerly chief chemist of Southwestern Portland Cement Co., El Paso, Texas, is now chief engineer of Peerless Cement Corp., Port Huron, Mich.

Continued on p. 29

The steadily increasing use of Flex-Plane Mechanical equipment, the general acceptance of Flex-Plane laboratory tested ideas in concrete slab laying, the library of more than 150 patents on concrete installing methods stand symbolic of the important role Flex-Plane has played, and is playing, in a continuous effort to provide better highways and airports the world over.

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(SEE HELTZEL STEEL FORM and IRON CO. AD ON PAGE 21)

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SEE PAGE 20—THE FLEXIBLE ROAD JOINT MACHINE COMPANY
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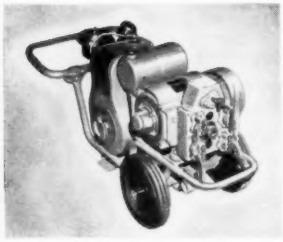

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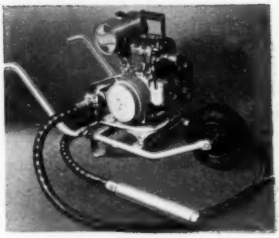

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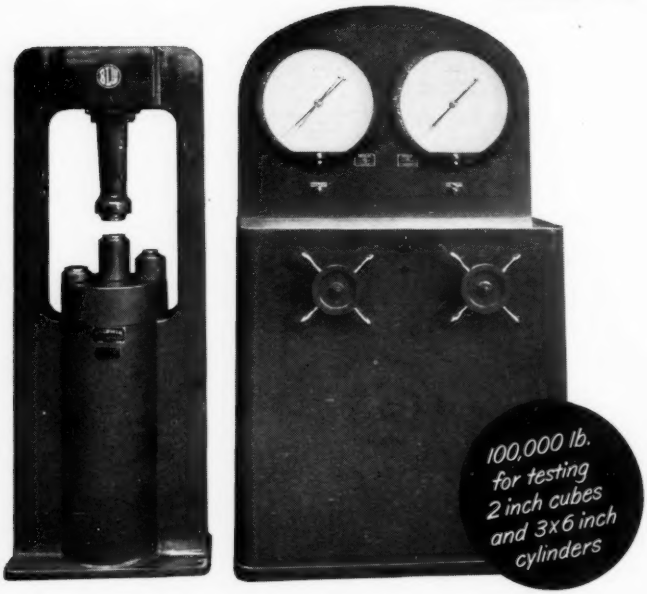

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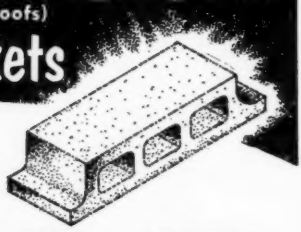

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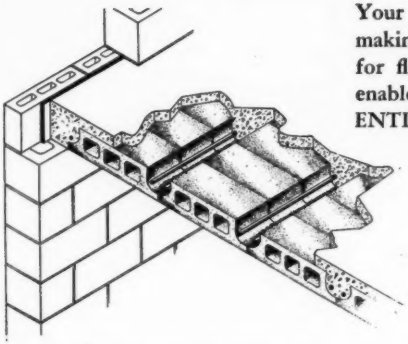

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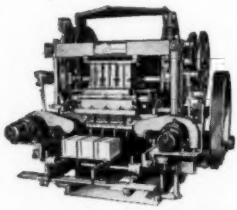

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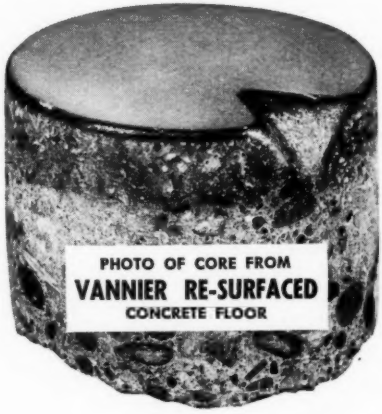

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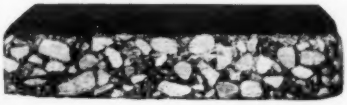

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Continued from p. 10

which the remainder were circularized regarding attendance and willingness to serve *if selected.*

By January 27 about sixty percent of the replies were in with a rather good surplus for choice in the interest of diversification. Some of the more distant prospects indicated inability to attend (as did some not so distant).

Actually the session promises to be a three-panel affair: the "experts," who will sit down front; the other "experts" who, for one or another of the several reasons mentioned, will be expected to perform from the audience as occasion offers; others of the ACI constituency which is, in fact, an aggregation of concrete experts.

With a wide assortment of questions already in, with others arriving almost daily, and a strong panel of fresh faces (some of which the leader has not yet seen), the session promises to take its place as a worthy successor to those previously held, and to be one of the convention highlights.

Continued from p. 19

Gilliland appointed general chemist

John L. Gilliland, Jr., formerly head of the chemical and cement laboratory of the Bureau of Reclamation, has been appointed general chemist for Ideal Cement Company's research laboratory, Fort Collins, Colo.

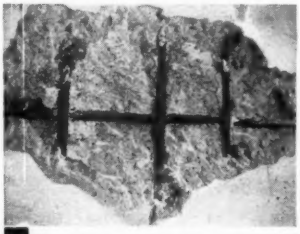
Mason contractors plan meeting

The Mason Contractors' Association of America, Inc., is planning its annual convention and exhibit to be held at The Palmer House, Chicago, Ill., February 24-27, 1952.

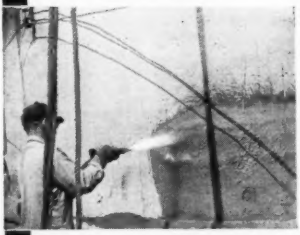
S. H. Westby, assistant manager, PCA Housing and Cement Products Bureau, will be one of the principal speakers. Mr. Westby, a specialist in concrete masonry construction, will speak on "A Modern Approach to Concrete Masonry."

Gerald Pickett

Gerald Pickett, formerly on the faculty of the Kansas State College of Agriculture and Applied Science, Manhattan, is now Professor of Mechanics, University of Wisconsin, Madison.



Typical example of spalling. Note corrosion of reinforcing rods exposed by disintegrating concrete.



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Preload Co. changes policy

By a complete revision of its operation, Preload Co., Inc., New York, N. Y., now offers consulting services to any client who wishes to use prestressing, employing its own unrestricted systems of prestressing as a yardstick against which proprietary systems may be measured, thereby permitting clients to obtain competitive bids from general contractors, who on their level may choose competitively from among several prestressing systems.

University of Illinois research on floor slabs

The University of Illinois Small Homes Council is continuing its research studies on floor slabs so that definite recommendations on the design of foundations for concrete floor slabs can be made. The performance of floor slabs is being studied to determine the relation between movement, weather, moisture and soil type—all essential factors in establishing design and construction requirements.

While it is customary in northern areas to extend the foundation of a basementless house below the frost line, the merits of this practice have never been subjected to research scrutiny. Since such construction is a major cost factor in a small house, it is believed that research might show how costs can be cut.

The study is being made under a research grant to the University by Levitt and Sons, Inc.

Clyde C. Pugh

Clyde C. Pugh, assistant engineer, Hazel-Atlas Glass Co., Wheeling, W. Va., died recently. After graduating from West Virginia University in 1912, he joined the staff of the Wheeling Traction Co. He was subsequently assistant engineer for the West Virginia Traction and Electric Co.; engineer and vice-president for the C. B. Kimberly Co.; and engineer with the C. C. Smith Co. From 1921-1935 he was a member of the firm of Conrad and Pugh.

Mr. Pugh had been affiliated with ACI since 1944.

Honor Roll

February 1—December 31, 1951

Sixteen additional members are listed on this month's Honor Roll with two or more credits—still less than 1 percent of the total Membership.

There is but a short time remaining to have your name listed with those instrumental in increasing ACI's membership. Credit for all members proposed in February is included in the current Honor Roll—there is yet an opportunity to be on the Honor Roll for 1951!

Newlin D. Morgan, Sr. (Ill.)	40
Newlin D. Morgan, Jr. (Mich.)	17½
Eddy Hernandez (La.)	10½
James A. McCarthy (Ind.)	10
J. E. Jellick (Calif.)	9
Oscar A. Nunez G. (Venezuela)	7½
Adolf A. Meyer (Tenn.)	7
Jose A. Vila (Cuba)	7
L. G. Farrant (Fla.)	6
H. J. Gilkey (Ia.)	6
Samuel Hobbs (Calif.)	6
Alvin C. Loewer, Jr. (Pa.)	6
Jacob Adler (Israel)	5½
Anthony G. Giardina (N. Y.)	5
Frank Kerekes (Ia.)	5
Alexander Klein (Calif.)	5
F. Thomas Collins (Calif.)	4½
Oscar Schreier (N. Y.)	4½
Oliver J. Julian (Mass.)	4
Ivan M. Viest (Ill.)	4
George D. Youngclaus (Calif.)	4
George E. Hatch (Hawaii)	3½
Fernan Rodriguez-Gil (Venezuela)	3½
Rafael Ruiz (Guatemala)	3½
C. H. Scholer (Kans.)	3½
Oscar A. T. Gimesy (Australia)	3
C. P. Siess (Ill.)	3
Curzon Dobell (N. Y.)	2½
E. L. Howard (Calif.)	2½
T. C. Kavanagh (Pa.)	2½
F. T. Mavis (Pa.)	2½
F. N. Menefee (Mich.)	2½
Donald Agrimson (S. D.)	2
Boyd G. Anderson (N. Y.)	2
J. F. Baxter (N. Y.)	2
E. J. Critzas (Mo.)	2
John H. Cullinan (Mass.)	2
Robert G. Deitrich (Md.)	2
James N. DeSerio (N. Y.)	2
Frank L. Ehasz (N. Y.)	2
Gerald K. Gillan (Mo.)	2
H. F. Gonnerman (Ill.)	2
H. R. Harbeson (Mich.)	2
H. E. Howe (Texas)	2
Henry L. Kennedy (Mass.)	2
David V. Lewin (Ohio)	2
R. B. MacNee (Calif.)	2
David R. McKenzie (Brazil)	2
Willlam McGuire (N. Y.)	2

Continued on p. 34

New Members

Membership now over 5500

The Board of Direction approved 63 Individual, 2 Corporation, 21 Junior and 11 Student Memberships for December. With adjustments for losses—resignations, deaths, nonpayment—the total membership reached 5510 on January 1.

ACKER, S. E., Montreal, P. Q., Canada (Chf. Engr., Dir., Creaghan & Archibald, Ltd.)
ALLEN, HOWARD F., JR., Catano, P. R. (Asst. Field Engr., Long Constr. Co.)
BELANGER, PIERRE, Quebec, P. Q., Canada (Design, Drafting, Detailing)
BOEGLIN, ALFRED R. (Jr.) North Andover, Mass. (Inspection, Thompson & Lichtner Co., Inc.)
BRADBERRY, CARROLL E., Los Altos, Calif. (C. E., U. S. Representative of Colcrete Ltd.)
BRADLEY, JAMES H. (Jr.) Monroe, La. (Struct. Engr., Design, S. E. Huey, Cons. C. E.)
BRANDT, ROBERT J., La Canada, Calif. (Job Engr., Bechtel Corp.)
BRILL, DONALD E., Williamsville, N. Y. (Design, Inspection, Field Supervision, J. N. DeSerio, P. E.)
BRODY, DAVID (St.) Detroit, Mich. (Wayne Univ.)
BUSER, WARREN M. (Jr.) Union City, N. J. (Struct. Design, Inspector, Parsons, Brinckerhoff, Hall & MacDonald)
BYCE, RICHARD C., JR. (St.) Ann Arbor, Mich. (Univ. of Michigan)
CALGI, RAYMOND (St.) New Rochelle, N. Y. (Columbia Univ.)
DALTON, ROBERT D., JR. (Jr.) Lafayette, Calif. (Struct. Designer, Robert D. Dalton, Cons. Struct. Engr.)
DAVIS, J. HENRY, Strawn, Texas (Engr., The Featherlite Corp.)
DILLINGHAM, GEORGE M., Philadelphia, Pa. (Sr. Engr., Turner Constr. Co.)
DONOHUE, WILLIAM VINCENT (Jr.) Cambridge, Mass. (Design, Drafting, Cleverdon, Varney & Pike)
DOULL, ROBERT M., Montreal, P. Q., Canada (Dir., Gen. Mgr., Canada Gunite Co., Ltd., Preload Co. of Canada, Ltd.)
DRDLA, STANLEY, Omaha, Nebr. (Chf., A. I. O. Strategic Air Command)
DUBIN, MILTON, Jamaica, L. I., N. Y. (Chf. Struct. Engr., H. K. Ferguson Co.)
DUICK, EMORY L., Skokie, Ill. (Designing, The Austin Co.)
DUNCAN, ADRIEN A., Mt. Holly Springs, Pa. (Sr. Engr., Capitol Engrg. Corp.)
DUPONT, M. F., Chicago, Ill., (Designing, Drafting, General Portland Cement Co.)
ESCOBAR B., DARIO (Jr.) San Salvador, El Salvador, (Engr., Direccion de Obras Hidraulicas)
FISK, PAGE, Long Beach, Calif. (City Chem., Testing Engr., City of Long Beach)
FLORIDA STEEL SUPPLY CORP., Hialeah, Fla. (Paul R. Greenfogel, Pres.)
FORDYCE, PHIL, Chicago, Ill. (Engr., Portland Cement Assn.)
FRISCH, EUGENE, Port Chester, N. Y. (Struct. Designer, Mianulli & Lamarca & Assocs.)
FRYAR, JOE B. (Jr.) Memphis, Tenn. (Designing, Drafting, Inspection, B. S. Merrill & O. C. Mann, Struct. Engrs.)
FRYSINGER, JAMES B., Pittsburgh, Pa. (Dist. Sales Mgr., Pa. Glass Sand Corp.)
GEARY, T. C., Redwood City, Calif. (Chf. Chem., Pabco Products Inc.)
GENTILE, JOSEPH, Buffalo, N. Y. (Mgr., Buffalo Dist. Office, Pittsburgh Testing Lab.)
GERACE, FRANCIS E., Hammond, Ind. (Asst. Constr. Supt., Morrison Constr. Co.)
GHINZBURG, EMANUEL, Ramat-Gan, Israel (Mgrg. Engr., Solel Boneh, Ltd.)
GRAY, VANCE J., Toledo, Ohio (Struct. Engr., Raymond C. Reese, Engr.)
HALTER, CHARLES D., JR. (Jr.) Chicago, Ill. (Designer, Draftsman, Chicago Dept. of Public Works)
HARRIS, LEONARD A., (St.) Urbana, Ill. (Univ. of Illinois)
HECKROTH, E. R., Wilmington, Del. (Designer, E. I. DuPont de Nemours & Co.)
HOELTJE, W. C. (Jr.) Chicago, Ill. (Research Asst., Grad. St., Univ. of Illinois)
HURLBUT, Co., F., Green Bay, Wis. (F. J. Lenfestey, Pres.)
HUTSELL, JOHN L., Everett, Wash. (Plant Super., Assoc. Sand & Gravel Co.)
INGRAM, JARRELL L. (Jr.) St. Louis, Mo. (Designing, Drafting, Terminal Railroad Assn. of St. Louis)
IVANAUSKAS, JUOZAS (Jr.) Chicago, Ill. (Designing, Drafting, A. J. Boynton, Engrs.)
JAROSZ, STANLEY, Brooklyn, N. Y. (Struct. Design, Ammann & Whitney)
JENTOFT, ARTHUR P. (St.) Bethlehem, Pa. (Lehigh Univ.)
JIMENEZ, DANIEL (St.) Ames, Ia. (Iowa State College)
JONES, ROBERT S. (St.) Chicago, Ill. (Univ. of Illinois)
KARPOWICH, DANIEL B., Drexel Hill, Pa. (Struct. Design, Day & Zimmermann, Inc.)
KHALILI, MOHAMMAD HOSEIN (St.) Iowa City, Ia. (State Univ. of Iowa)
KROEGER, OTTO P., El Paso, Texas (Chf. Chem., Southwestern Portland Cement Co.)
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LEWIS, GORDON V., Columbus, Ohio (Concrete Tech., Sales Engrg., W. E. Anderson Sons Co.)
LONG, HORACE T., Selah, Wash. (Design, Bidding, Constr. Super., W. M. Yeaman, Engrs. & Contractors
LOWELL, RICHARD A. (Jr.) San Francisco, Calif. (Drafting, Designing, Pacific Gas & Electric Co.)
MARTIN, LEWIS EDWARD, Martinsburg, W. Va. (Chf. Engr., Genl. Supt., W. Harley Miller, General Contractor)
MATHER, KATHARINE, Jackson, Miss. (Chf., Petrography Sect., Corps of Engrs.)
MATTHEWS, ARTHUR J., III (Jr.) Lincoln Park, Mich. (Drafting, Struct. Design, Giffels & Vallet, Inc.)
MAYER, GUSTAV, Shaker Heights, Ohio (Struct. Engr., Trygve, Hoff & Assoc.s)
MHOON, REX E., Long Beach, Calif. (Supervision, Inspection, Albert C. Martin & Assocs.)
MIYASHIRO, LARRY S., (Jr.) Honolulu, T. H. (Designing, Drafting, Corps of Engrs.)
MURPHY, ROBERT J. (Jr.) Lawrence, Mass. (Drafting, Designing, Cleverdon, Varney & Pike, Cons. Engrs.)
OFFUTT, J. S., Chicago, Ill. (Merchandise Mgr., U. S. Gypsum Co.)
PALAS, TABARE, Montevideo, Uruguay (C. E., Public Works Ministry)
PATTISON, WILLIAM E. (St.) Kansas City, Mo. (Univ. of Missouri)
PENNY, J. M., Emporia, Kans. (Mgr., Ready Mixed Concrete Co.)
PERKINS, HARRY B., JR., Plainfield, Ind. (Designing, Drafting, Supervision, Inspection, A. J. Boynton & Co.)
PHILLIPS, GLENN H., Dallas, Texas (Assoc., Office Engr., Zeigler, Houseman & Assocs.)
RAE, WILLIAM M., Winthrop, Mass. (Plant Engr., Boston Sand & Gravel Co.)
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Continued on p. 34

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Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Soil compaction

Detailed information, including useful new production data, for figuring work schedules and costs on soil compaction jobs is contained in Bulletin No. 621 just published by Barco Mfg. Co., Chicago, covering the use of the Barco portable gasoline "Rammer" for tamping fill or back-fill in restricted areas. Of special interest to builders, contractors, supervisory engineers, and project managers is a discussion of the use of "soil compaction" as a means of attaining (1) greater permanence, (2) lower costs, and (3) earlier completion dates on many construction projects.

Applications described in the new bulletin include use of high-degree soil compaction in construction work on highways, roads, streets, buildings, dams, bridges, airports, landing strips, ordnance proving grounds, pipe lines, utility trenches, parks and cemeteries.—Barco Manufacturing Co., Dept. R-31, 1801 Winnemac Ave., Chicago 40, Ill.

Synthetic rubber resin based coatings

A brochure released by Casey and Case Coating Co. presents characteristics, properties, uses and methods of application of synthetic rubber resin based coatings. Damp-wall enamels, stucco-masonry coatings, floor finishes and machinery enamels are described.—Casey and Case Coating Co., P. O. Box 151, Maywood, Calif.

Scaffold jack

An 8-page, two-color booklet describes the use of the WACO scaffold jack. Ask for form KP-B.—Wilson-Albrecht Co., Inc., 3565 Wooddale Ave., Minneapolis 16, Minn.

Conveyor and elevator belting

New York Belting and Packing Co. has published a new catalog on conveyor and elevator belting which gives all the necessary data to lay out a drive or specify a belt.

Complete tables on carrying capacities, horsepower factors, pulley diameters, maximum and minimum plies for proper troughing and other engineering information are included.

A copy of the catalog may be obtained upon request.—New York Belting and Packing Co., 1230 Avenue of the Americas, New York 20, N. Y.

New Members

Continued from p. 31

San Martin, Angel, Havana, Cuba (Engr.)
Schlenker, Norman E., Baldwin, L. I., N. Y. (Designing, Checking, Supervising, Seelye, Stevenson, Value & Knecht)
Schwartz, Jules, Cleveland, Ohio (Struct. Engr.)
Serafim, Joaquim Laginha, Lisbon, Portugal (Head, Dams Studies Sect., Laboratorio de Engenharia Civil)
Singleton, William C., Newark, N. J. (Service Engr., Lehigh Portland Cement Co.)
Sitts, Harold F., Tacoma, Wash. (Bridge Engr., Design, Pierce County Road Engr.)
Skilken, B. Lee (St.) Columbus, Ohio (Ohio State Univ.)
Smith, Estel E., Ste. Genevieve, Mo. (Constr. Supt., Mississippi Lime Co.)
Staley, Leo D. (Jr.) Munster, Ind. (Draftsman, A. J. Boynton & Co.)

Tancredi, Lawrence A. (Jr.) Hollis, Queens, N. Y. (Design, Detail R. C. Work, Byrnes Assocs.)
Tashiro, Noboru, Denver, Colo. (Struct. Engr., R. J. Tipton & Assocs., Inc.)
Teng, Hsi-Huan (Jr.) Berkeley, Calif. (Design, Kaiser Engrs.)
Tewfik, Mohamed Mohsen, Ithaca, N. Y. (Instructor, Fouad I University)
Tiarks, F. Alan (Jr.) St. Louis, Mo. (Products Engr., Granco Steel Products Co.)
Tilbury, Charles O., Jr., (Jr.) Houston, Texas (C. E., Ambursen Engrg. Corp.)
Tooker, Harry L., Houston, Texas (C. E., Ambursen Engrg. Corp.)

Viger, J. W., Mt. Prospect, Ill. (Designing, Drafting, S. F. Nydam)

Wingo, Allan, Birmingham, Ala. (Sales Engrg., Alabama Cement Tile Co.)
Woodruff, Joseph Alexander, New York, N. Y. (Constr. Engr., Corps of Engrs.)

Yolles, Morden S., Toronto, Ont., Canada (Cons. Engr.)

Zetlin, Lev, Ithaca, N. Y. (Research Assoc., Cornell Univ.)

Honor Roll

Continued from p. 30

Thomas J. Reading (S. D.)..... 2
Dana A. Scheid (Ind.)..... 2
Howard Simpson (Mass.)..... 2
Harry F. Thomson (Ill.)..... 2
LeRoy A. Thorssen (Canada)..... 2

Captain Raymond Harvey Medal of Honor

THE 17TH INFANTRY REGIMENT was attacking Hill 1232 near Taemi-Doug, Korea. Charlie Company, Captain Harvey commanding, was moving up when dug-in Red guns pinned it down. Calling for covering fire, Captain Harvey advanced *alone*, wiped out four machine gun emplacements. He caught a bullet through the lung. But he stayed in action until the ridge was won.

"In Korea," says Captain Harvey, "we stopped aggression by *united* strength. *You* were helping—every time you bought a Defense Bond. Because your Bonds were doing more than just helping keep you financially stable. They were backing *us* up in the field with *American production power.*

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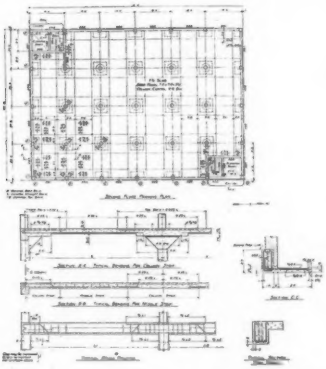
Manual of Standard Practice for Detailing Reinforced Concrete Structures

(ACI Standard 315-51)

for the

Designer
Detailer and
Student



$3.00 (to ACI Members—$1.75)

Completely revised to include new-style A-305 deformed reinforcing bars, higher bond values permitted by the ACI Building Code, other changes to conform more closely to the ACI Code, and editorial changes for greater clarity.

The fundamental purpose of all "engineering drawings" is to enable the builder to transform the designer's calculations and design into the structure he has conceived. These drawings give the information necessary for building forms and placing concrete and for preparing the "placing drawings" (sometimes called "detail drawings") which must contain the information necessary for fabricating the reinforcement and placing it in the forms.

The **Manual** illustrates typical engineering and placing drawings prepared according to the standards. The designs are not intended as standard designs, although in general they meet the requirements of the American Concrete Institute's "Building Code Requirements for Reinforced Concrete" (ACI Code). They are intended to show **standard detailing methods.**

American Concrete Institute 18263 W. McNichols Rd. Detroit 19, Mich.

The Detailing Manual includes—

Drawing standards—Standards and comments on size of sheet, layout, scales and symbols.

Marks—Discussion of systems of marking members and reinforcement.

Schedules—Discussion of types of schedules and recommendations for their use.

Engineering drawings—Information required on drawings and best methods of showing it.

Placing drawings—Best methods of showing necessary information.

Fabricating detail standards—Standards for bending, hooks, stirrups, typical bar bends, slant lengths, beam widths, spirals, ties, column verticals, splices and dowels.

Notes to designers and detailers—Attention is called to points which frequently are overlooked or which cause trouble. Conformance with the suggestions given eliminate most difficulties of interpretation, fabrication and construction.

Shop procedure—Brief description of steps in the shop.

Warehouse stock—Notes material usually carried in stock.

Tolerances—Standard tolerances in dimensions of fabricated reinforcement.

Extras—Basis for extra charges for material and fabrication.

Welded wire fabric—Data on fabrication, size limitations, detailing and common styles.

Accessories—Standard accessories and their use.

Typical drawings—21 typical engineering and placing drawings are shown for various structures to illustrate the use of the standards and methods advocated in the manual. A short discussion of the important points accompanies each drawing.

Typical drawings

The engineering drawings used in the manual were selected from plans of actual structures but have been modified in some details to conform more closely with the ACI Code and to illustrate recommended methods of presenting the information. Shop prepared placing drawings accompany each engineering drawing.

- Concrete joist floor
- Flat slab floor
- Beam and girder floor
- Two-way slab and beam floor
- Foundations
- Columns
- Architectural concrete building
- Wall details
- Stair details
- Circular tank
- Rigid frame bridge

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*All members in good standing Mar. 1, 1952 will receive a card entitling them to purchase an initial copy for $1.00.

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DISCUSSION

Discussion closed January 1, 1952

Sept. Jl. '51

Earthquake Resistant Design Considerations—R. R. Martel

Comprehensive Numerical Method for the Analysis of Earthquake Resistant Structures—Charles S. Whitney, Boyd G. Anderson and Mario G. Salvadori

Multistory Buildings Designed to Resist Earthquakes—John J. Gould

Lightweight Concrete for Lower Construction Costs—J. A. Murlin

Designing for Continuity in Prestressed Concrete Structures—Alfred L. Parme and George H. Paris

Pumice—Lightweight Aggregate—Leslie I. Neher

Hydration Products Formed in Cement Pastes at 25 to 175 C—George L. Kalousek and Milton Adams

New Techniques in the Study of Setting and Hardening of Hydraulic Materials*

By J. CALLEJA†

SYNOPSIS

This paper attempts to establish basic ideas for research on the setting and hardening of hydraulic materials through use of methods based on the variation of the electrical resistance of a paste during its setting period.

This method is superior to classical ones, due to its wider application and provision for more detailed data on the phenomenon. Moreover, it can be applied automatically.

Theoretical considerations as well as experimental evidence show that the beginning and end of the setting period are indicated on resistance-time graphs by well defined points closely linked to points of graphs showing temperature changes during setting.

INTRODUCTION

The beginning and end of the setting period of a cement has long been determined by Vicat's classical test. This test establishes and implies a relationship between the physicochemical process of the system under investigation and the development of its mechanical properties.

However, if we define setting as the whole group of physicochemical transformations undergone by a hydraulic material from the moment it comes in contact with water until transformations practically cease, then the relationship implied by Vicat's test is justified only in its qualitative sense. Moreover, Vicat's test tells nothing about the intermediate changes and properties.

For these reasons, there is need for a new technique which will facilitate the study of the setting and hardening of hydraulic materials, and satisfy the following requirements: (a) the empirical nature of Vicat's method should be avoided as far as possible, (b) it should supply more exact and complete data than has been obtained so far. It should facilitate the study of variables which can affect the setting phenomenon, and (c) the probable causes of personal error should be eliminated by automatic functioning.

With these conditions in mind, the study of changes of some electrical variables, depending on the phenomenon under investigation, was undertaken. Among the electrical variables studied were dielectric constant, electrical resistance and conductivity and electrode potentials. The elec-

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trical resistance was selected for the first set of experiments. The other two were not neglected, although not considered at this time.

PREVIOUS STUDIES

There are very few references to electrical conductivity of cements during setting. The first worth mentioning is Bogue's reference[1] to Shimizu.[2] Shimizu states that the investigation of the setting of cement is more accurate if done by electrical rather than by mechanical and thermal methods. He concludes that the end of setting is given by an inflection point in the conductivity-time graphs, when working at a constant temperature.

Baire[3] investigated the effect of passing an electric current across a cement-water paste and assumed that setting commenced as soon as there was a large drop in the reading of an ammeter. Results were not accurate, as the setting was accelerated because of heating due to the Joule effect.

Michelsen[4] attempted to investigate the beginning and end of setting by considering the galvanic currents generated in any paste between two different metals. On voltage-time graphs the beginning is given by the point at which a constant section of curve commences to slope upwards and the end by the point corresponding to the maximum voltage.

Dorsch[5] also investigated some variable electrical properties and behavior of cement pastes in connection with the setting phenomenon.

Kleinlogel and Hajnal-Konyi[6] consider the methods of Vicat and Gillmore insufficient and give reasons for adopting thermal methods. Jesser,[7] who considered setting and hardening as an electrostatic phenomenon, studied the emf of galvanic elements in various cements in connection with the coloidal nature and changes in the pastes.

Boast[8] investigated conductometric methods for studying several problems related to cement pastes and mortars. Among them, he studied the setting phenomena during the first three hours and tried to correlate conductivity with the 28-day compressive strength. Chisholm[9] showed in new data the ineffectiveness of Vicat's method for certain types of cements and hence the need for new techniques.

EXPERIMENTAL WORK

The tests were made under normal laboratory conditions, although the system permits observations to be made conveniently under water. Test pieces were relatively large and no isothermal conditions were maintained. On the contrary, since setting cannot be isothermal in practice, as the reactions involved are exothermal, experiments were made in almost adiabatic conditions which generally govern in practice. Changes in temperature of the paste were studied at the same time as the variation in electrical resistance.

Electrical resistance in the first experiments was measured by taking the ratio of the potential difference, applied at each moment to the electrodes, and the strength of the current across the paste. Alternating current was

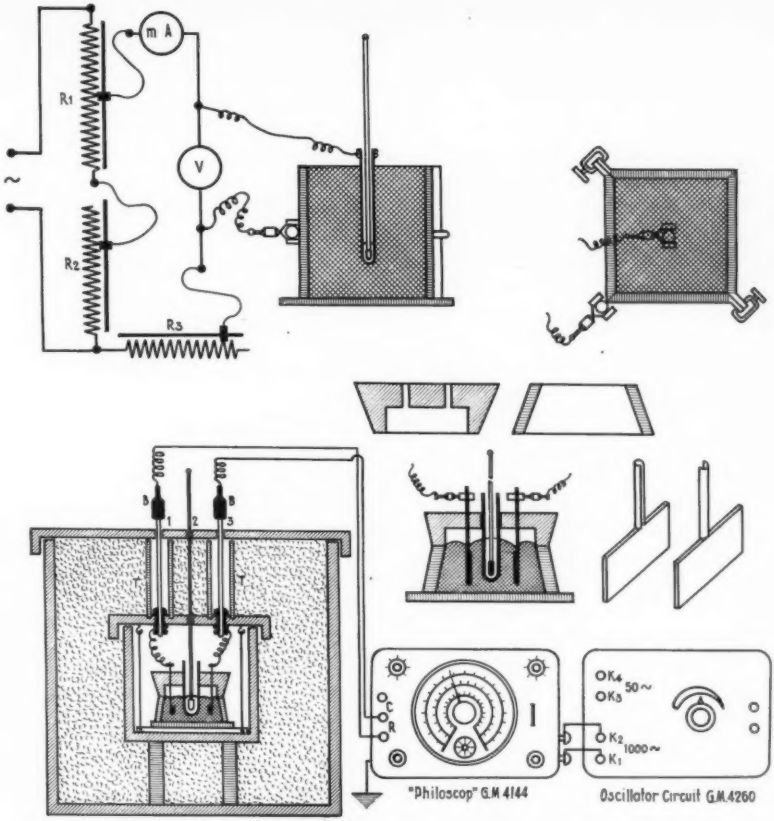

Fig. 1. Top: Preliminary form of apparatus for measuring electrical resistance of paste Bottom: Final form of apparatus

applied so as to avoid electrolytic and polarizing effects. The intensity across the test paste was but a few milliamperes and passed only during the moment of making the measurements to prevent heating by Joule effect.

In each case the paste, mixed with a given amount of water, was put into a 7 x 7 x 7-cm cubic iron mold and placed on a glass plate. The mold serves as one electrode, the other being an iron tube containing some mercury, introduced at the center of the paste. In this tube fits a thermometer for measuring the temperature changes in the paste. No precautions were taken to insure adiabatic conditions. The apparatus and electrical diagram are shown in the upper part of Fig. 1.

The method adopted, after modifications, includes a device to insure almost adiabatic conditions during the setting process. The container is a double-

walled box, (Fig. 1, lower part) the space between walls being filled with cork or other insulating material so as to completely surround the test mold. The external and internal lids have openings through which pass the electric leads and thermometer or the wires of the thermocouple as the case may be. The leads are loosely connected at the lower end and at the lower part of the internal lid to two cables provided with clamps, to which the electrodes immersed in the cement paste in the mold can be attached. The contact is made from outside by pushing in the long, *BB*, contacts. The cardboard tubes, *TT*, (Fig. 1) affixed to the lid of the inner box make it easy for the electric leads to pass through the insulating material.

Having prepared the paste beforehand the mold is placed on a glass slab which rests in turn on a sheet of cardboard, *S*. This sheet can be suspended by four supports, as shown in the lower part of Fig. 1, once all internal contacts are complete. This facilitates removal of the whole assembly from the insulating box.

The mold is a truncated cone made of an insulating material. The electrodes are two rectangular iron plates of suitable size, placed opposite each other at a convenient distance apart. The distance as well as the height of the electrodes within the paste are maintained by fixing both electrodes inside a cork cover of the same size as the mold area. The lower part of the cover is hollowed out over the paste so that any paste that overflows the mold when the electrodes and other accessories are placed in it has space in which to move. The cork cover also supports the iron tube containing mercury in which is placed a thermometer permitting 0.05 C measurements.

Resistance was measured by Philips conductivity gages types G.M. 4140 and G.M. 4144 (Philoscop I and II) having a cathode ray indicator to balance the resistance to be measured. With these universal bridges, work was done either at 50 or 1000 cycles. In the latter case a low frequency Philips type G.M. 4260 oscillator was used. It was found convenient to work with a frequency of 1000 cycles as this provides a greater measuring sensitivity. This is so because in addition to the resistances between the plates fixed in the paste, the electrodes and the paste act as a condenser. In any case, for measurement, there is a resistance wired in parallel with a condenser (condenser with losses). In such a system the capacitance may disturb the resistance measurement, but this disturbance is at least partially eliminated by working at high frequencies.

The results agree fairly well with those obtained by the method first described, but the measurements are more accurate with the Philoscop, as the sensitivity can be regulated easily at any time. Measurements with the potentiometric arrangement make it necessary to take simultaneous readings of two apparatus. The lower part of Fig. 1 shows the assembly.

TEST RESULTS

Discussion is limited to the results obtained with respect to the beginning and end of setting of the paste. Fig. 2 shows some of these results ob-

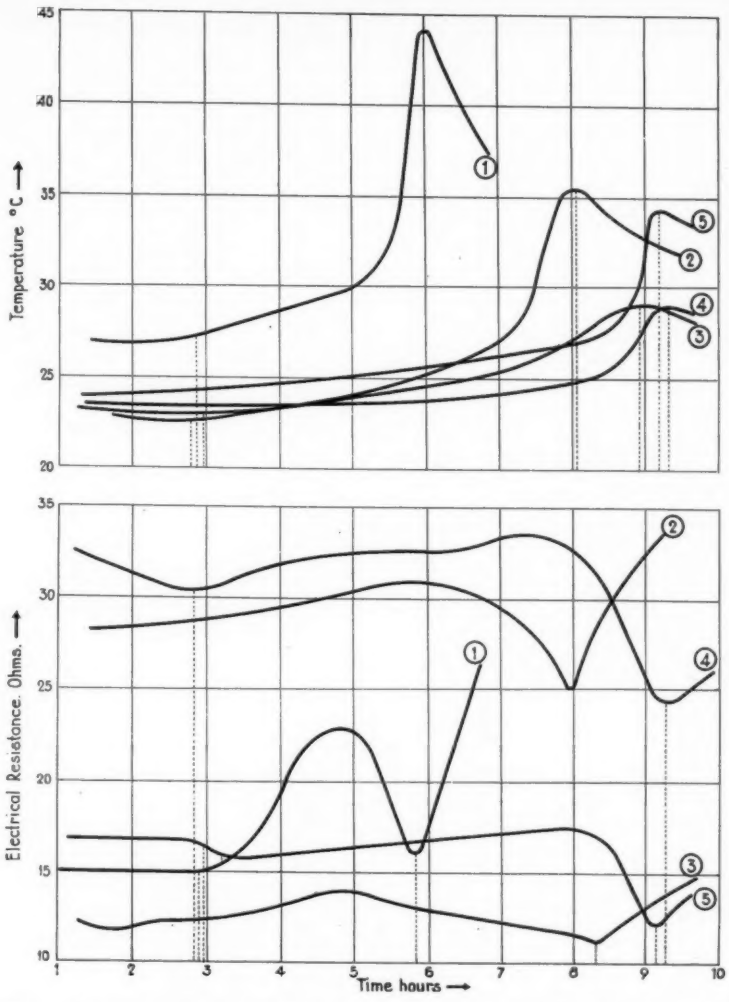

Fig. 2. Top: Temperature-time (T-t) curves obtained with cement pastes during the setting process
Bottom: Electrical resistance-time (R-t) curves of the same pastes

tained by employing the technique described. The upper part presents temperature-time (*T-t*) curves. In the lower part of Fig. 2 the electrical resistance-time (*R-t*) curves, of the corresponding experiments, during the setting of different portland cement pastes can be seen.

The *T-t* curves show two well-defined points, indicated by dashed lines, for each curve in the graph. The first on the left is the point at which the temperature begins to increase slowly but continuously; the second on the right, is that of maximum temperature.

R-t curves also show two well-defined points. That on the left is the point at which the resistance increases more or less slowly (curves 1, 2 and 3), or eventually shows a slight dip as in curves 4 and 5. The second point on the right is in each case a very well-defined minimum (Fig. 2).

Also noticeable is the relationship between the abscissas of the points in both types of curves. As an almost general feature, the maxima on *T-t* curves appear somewhat displaced toward the right in relation to the minima on the *R-t* curves, although always exhibiting the same relationship.

The first and second points in the *R-t* curves give the times at which the setting begins and ends, respectively. This will be discussed in detail later. *R-t* curves also have a maximum, the height and position of which is variable and seems to depend on secondary experimental effects.

For greater simplicity the *R-t* curves have been drawn without correcting the values of *R* for difference in recording temperature, *i.e.*, for every increase of 1 C in the temperature, the conductivity increases by 1 percent and the electrical resistance decreases correspondingly.

This correction has no influence on the shape of the *R-t* curves, nor on the position of the characteristic points indicating the beginning and the end of the setting process. The correction merely affects the absolute value of the resistance, which in this case is only of secondary interest.

As regards the precision, sensitivity and reproducibility attained with the electric method to determine the setting time of a cement, Fig. 3 and 4 show curves for experiments made under conditions as comparable as possible with the same cements employed to obtain curves 1 and 2 in Fig. 2, respectively, under somewhat different experimental conditions. The same results were obtained in several experiments with cements corresponding to curves 3, 4 and 5 and some others not recorded in the graphs.

The largest discrepancies registered, both with regard to the beginning and end of setting, are within a maximum variation of fifteen minutes. This is for tests under comparable conditions, and it is not an excessive range of variation considering the experimental circumstances.

The lower parts of Fig. 2 and 3 give the *R-t* curves for each of the various cements investigated. Not only for different cements, but also for the different variants of a given kind of cement, the corresponding curves are distinct, and each has a characteristic shape which can be easily reproduced.

DISCUSSION

Leaving for the moment the various theories for the physicochemical process of setting, assume a given cement mixed with a fixed proportion of water. Then in the process of setting, certain hydrates (formed from the

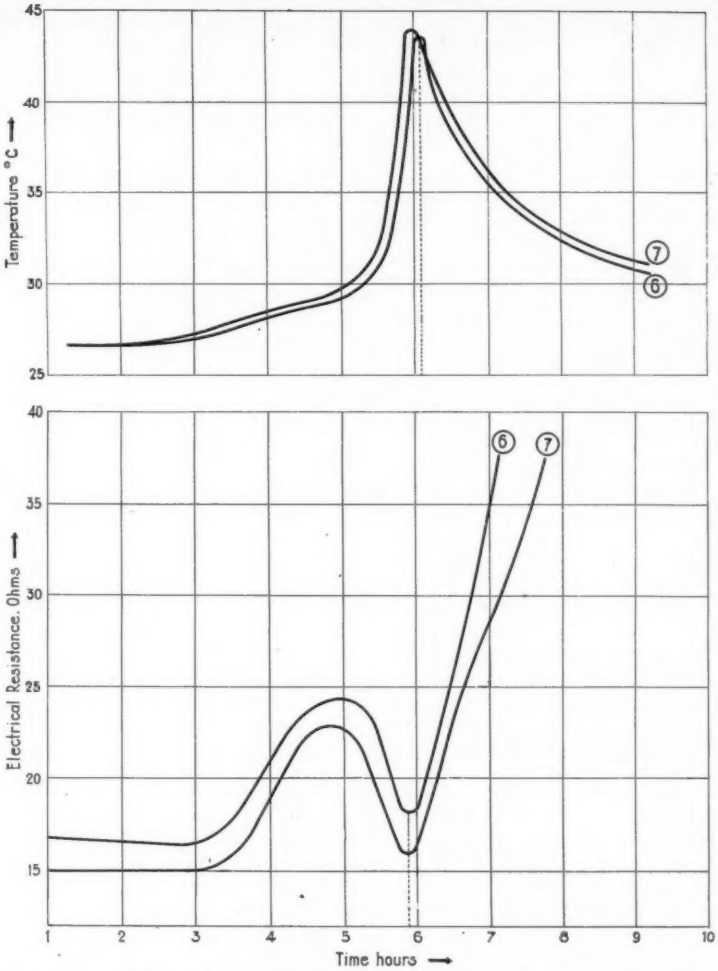

Fig. 3. Top: Additional T-t curves obtained with cement of 1 Fig. 2, to indicate reproducibility of results
Bottom: R-t curves of the same cement

corresponding anhydrous products which have previously passed into the solution) crystallize. In this case the solubility of the former is at every instant less than that of the latter. Hence, when the solution is saturated with anhydrous products it is over-saturated with hydrates; consequently these crystallize. Therefore it is easy to show graphically the variation of electrical resistance from the time the paste is made until after hardening.

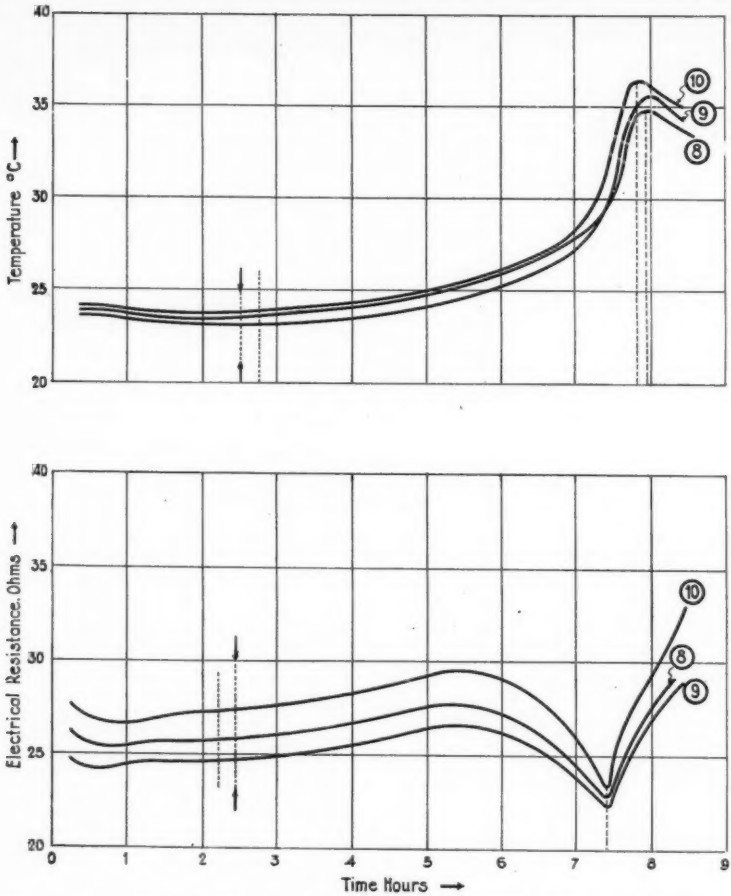

Fig. 4. Top: Additional T-t curves obtained with cement 2 of Fig. 2, to indicate reproducibility of results
Bottom: R-t curves of the same cement

Initially, the contact of the anhydrous components with the water will cause the solution or hydrolysis of the more soluble or hydrolizable parts of it (alkalies, etc., in the case of portland cement). This will result in a paste because of the relatively small amount of water normally used. There will also be a solution of a certain ionic concentration.

The electrical resistance of the heterogeneous system of the cement paste is due to two forces. One is the ionic conductivity of the solution, which depends on ionic concentration, temperature and type of ions present in the solution.

The other factor which can be interpreted as electrical resistance of the "ohmic" type, is due to increasing mechanical resistance and cohesiveness as setting progresses. As the system is heterogeneous, its electrical conductivity depends on the changes which both solid and solution phase undergo. These changes are closely linked to each other since the composition and concentration of the "solution" depend on the residual water available, while the composition and texture of the solid phase depend on the water taken up in combination. The sum total of these two quantities of water is the water added to the mixture, neglecting the water lost due to evaporation. Hence it is obvious that the two factors which affect the resistance of the system are almost inseparable. However, their distinct nature must be emphasized.

From the time the mixing of the paste is finished until setting becomes noticeable, the ionic concentration of the solution may increase due to hydrolysis of alkali-bearing compounds and perhaps aluminates which cause an increase in ionic conductivity. The texture of the solid phase shows little change during this period, and hence the electrical resistance of the system may drop or remain almost constant, until the time when setting begins.

During the process of setting, the products of hydration crystallize and separate from the solution. New anhydrous products will join the solution to re-establish equilibrium. This process will continue until the majority of the anhydrous components have gone into the solution and the majority of the hydrated components have crystallized.

As soon as hydrated compounds begin to crystallize, the amount of water which combines with solids increases. Hence, the remaining water in the solution decreases, and the texture of the solid phase begins to be modified. Within the solution, concentration and viscosity may also increase, so that freedom of movement of the ions will be hindered. Rigidity also increases in the solid phase. Therefore, as soon as the process of setting begins, the two factors which affect the resistance in the system will begin to increase it.

As soon as setting is finished the paste begins to harden. Evaporation and absorption continue, and from this point on the ohmic resistance increases at the same time as the ionic conductivity decreases. As a result, at the end of the setting process a new increase in resistance will be observed, even more noticeable than the first one.

Resistance-time (R-t) curves may take the form shown in the lower part of Fig. 5-I in which points A and B represent beginning and end of the setting process, respectively.

Ideal conditions for observation must exist. That is, it is essential that the only change in the electrical resistance of the system be due to its physicochemical properties. However, some extraneous influences cannot be left out of consideration; for instance, the temperature. During setting of a cement, a certain quantity of heat is given off which, unless removed, will increase the temperature of the setting mixture. This increase tends to reduce the electrical resistance, since it increases ionic conductivity.

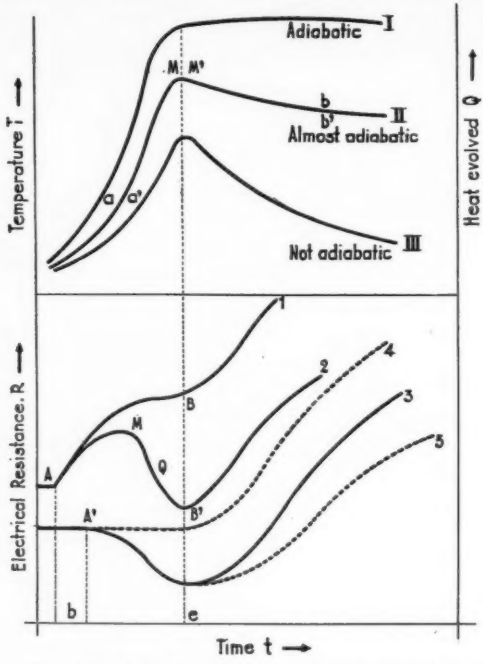

Fig. 5. Top: Analysis of typical T-t curves Bottom: Analysis of corresponding R-t curves

Consider how temperature changes during the process of setting. Under adiabatic conditions the curve defining the quantity of heat given out by a cement during hydration has the form shown in the upper part of Fig. 5-I, and it tends asymptotically towards a constant value, as time increases. If conditions are not adiabatic, there is a maximum value (curves II and III).

Actually, the temperature-time curves, under adiabatic conditions, also tend asymptotically towards a constant value. If conditions are nonadiabatic, then, as in the heat-time curve, a maximum value is shown.

The maximum values for heat generated and for temperature will have the same time coordinate in near adiabatic conditions. Hence, curves in Fig. 5 may represent either heat evolved, Q-t, or temperature, T-t, at the corresponding ordinate.

The Q-t or T-t curves showing the amount of heat given off due to hydration, or the temperature reached at each moment, indicate the progress of setting up to the moment considered. It is reasonable to assume that this phenomenon is finished when there is no further appreciable amount of heat given out. That is, when the end of the steep part of the adiabatic curves is reached, or in cases of the near adiabatic curves, when the maximum point has been attained, *i.e.*, when point M is reached in curve Q-t or M' in T-t. Similarly, the beginning of setting will be that instant when the steady increment, even if slow, of these curves begins.

Because of their shape, the influence of temperature on the electrical resistance-time curves can be readily appreciated. The maximum value of the temperature curve will probably correspond to a minimum value of the resistance curve. The downward sloping section b' of the temperature curve will correspond to an upward section in the resistance curve. The matter is more complex in the section at a'. On the one hand, the resistance tends to increase due to the progressive hydration and crystallization, although this increase probably proceeds at a diminishing rate (section A-B of curve I, Fig. 5). On the other hand, the resistance should diminish as the temperature rises as in section a'.

Hence, section A-B will be distorted, for, as the second effect becomes greater than the first, section A-B begins to decrease. Therefore it will have a maximum value at that point. When point M' in T-t is reached, section A-B is a minimum, and from then on there will be an upward sloping section, which will correspond to the downward section b'. From this it is possible to appreciate the coincidence, *i.e.*, the same abscissa for the maximum M' value in the curve T-t, and the minimum value B' in the R-t curve.

The maximum point M in curve R-t, has no particular significance physically. It merely indicates the point at which the tendency of the resistance to diminish, as a consequence of the increasing temperature, becomes equal to its tendency to increase due to the process of setting itself.

In view of the close relationship between the R-t and T-t curves, it appears justifiable to assume that the beginning and end of setting is defined by the abscissas of the points on the R-t curve.

The displacement of the maxima of T-t curves with respect to the minima of R-t curves seems logical, if it is remembered that the method of measuring the electrical resistance makes it possible to record changes instantaneously. However, in recording temperatures this is not the case. It takes some time for thermal equilibrium to be established between the changing system and the instrument which records the temperature. This fully accounts for the relative displacement of the two curves. In this sense, the readings for the R-t curves are more precise than those of the T-t curves, at least in this particular case, considering the method used to arrive at the T-t curves.

Concerning the precision of the method, the range of variation observed in the beginning and the end of setting is not excessive. For instance, the results can be influenced by the percentage of water in the mix, the adiabatic or nonadiabatic conditions of the test, the quantity of paste used, the dimensions and separation of the electrodes (this determines whether the electrical capacity or the ohmic resistance have the greater influence and hence, the way in which the measuring sensitivity is affected), and the frequency of the current used.

There are other variable factors worth considering, though not so easy to control. These include the time of the year and the place where the paste is produced, since these influence the temperature of the water used in the mix and atmospheric temperature and humidity (all setting tests were done in

the open air). Another factor is how much the paste has been worked, or, to put it another way, the skill of the operator. For the moment there is no mechanical means free from the personal factor which insures that paste is uniformly well mixed.

All these variables, controllable or otherwise, account for the variation in results, very small in any case, with each kind of cement. This also indicates the sensitivity of the method. In view of the remarks just made, other methods may appear to be more accurate because they are less sensitive.

Conforming to the prescribed conditions, the method described seems to be more accurate and sensitive than any of those being used at present to determine the beginning and end of the setting process. This is confirmed by the curves for tests No. 6 and 7 which were conducted simultaneously (Fig. 3). These tests were performed under similar conditions to demonstrate the precision and reproducibility of results by this method.

CONCLUSIONS

1. Classical methods for observing the setting process of hydraulic materials are inadequate as regards the amount and accuracy of the data supplied.

2. New techniques must be considered not only for the investigation of setting, but also of hardening of cements and similar materials. Such techniques can be based on the variation of electrical properties with time.

3. Among such properties, this paper has considered the electrical resistance of the cement paste. Both from theoretical considerations and experimental results reported here, it can be established that the time at which setting commences and finishes is indicated in the R-t curves by well-defined points.

4. R-t curves are in good agreement with the T-t curves.

5. Results obtained by standard methods are in fairly good agreement with the method proposed. This new method is superior to classical methods in accuracy, sensitivity, uniformity of results, and in the possibility of further new application, both as a new technique in the field of pure research, and as a possible standard method for measuring the setting of cements.

6. In any case, this method lends itself to automatic graph recording of the setting process under investigation.

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Discussion of a paper by J. Calleja:

New Techniques in the Study of Setting and Hardening of Hydraulic Materials*

By E. H. WATERS and AUTHOR

By E. H. WATERS†

The author claims that his method of determining setting time of hydraulic materials by observing variations of electrical resistance of the paste with time is superior to those presently used. This superiority is claimed chiefly on the grounds that: (1) The method has a sound theoretical basis (conclusion 3, p. 536). (2) The method shows greater accuracy, sensitivity and uniformity of results than do classical methods (conclusion 5, p. 536).

THEORETICAL BASIS

Justification for the claim that the method has a sound theoretical basis appears to rest on the points made by the author on pp. 530-535, where he treats qualitatively the probable effects of the setting process on the conductivity of the paste. From these he concludes that, under isothermal conditions, the resistance-time (*R-t*) curves would take the form of curve 1 in Fig. 5, *i.e.*, a more or less steeply rising curve with an approximately horizontal point of inflection at the time of final set. However, under practical conditions the rise in temperature of the paste as the setting proceeds will reduce its resistance and the theoretical point of inflection will appear as a minimum.

It is difficult to reconcile these conclusions with the test results (p. 530) where it is said that "for greater simplicity the *R-t* curves have been drawn without correcting the values of *R* for difference in recording temperature . . . This correction has no influence on the shape of the *R-t* curves nor on the position of the characteristic points indicating the beginning and the end of the setting process." (This argument and the following assume that in referring to correction for differences in temperature the author is considering the effect of the temperature rise during setting and not merely the small differences in initial temperature.)

If the former statement (that isothermal *R-t* curves would have the form of curve 1, Fig. 5) is correct, then the minimum observed is simply a reflection of the temperature rise and the new method only a cumbersome way of per-

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forming the old established technique of determining setting time by observing the temperature rise of the paste. If the latter statement (that temperature corrections do not affect the shape of *R-t* curves) is correct, then curve 1, Fig. 5, would appear to be meaningless. A little calculation from the published data shows that either form of corrected curve can be obtained, depending on the assumptions made regarding temperature coefficient.

It is difficult to know just what value to assign to the temperature coefficient of resistance in such a material; Mr. Calleja gives 1 percent per degree C but he does not say what is the basis of his figure. Since conductivity of the paste must be almost wholly ionic (one would expect dry portland cement to have an extremely low conductivity of the same order of magnitude as those of other silicate minerals*) it appears reasonable to assume that the temperature coefficient of resistance would be similar to those of solutions of bases and salts. The temperature coefficient for such solutions varies with the solute and its concentration. For dilute solutions it lies between 1.8 and 2.2 percent per degree C.[1] For more concentrated solutions it usually rises to about 2.4 percent per degree C[2] and with a few materials to as high as 3 and even 4 percent per degree C. It would be interesting to know whether Mr. Calleja has any experimental or other justification for his value of 1 percent and if so why it is so much lower than values normally found for electrolytes.

If curve 6, Fig. 3, is corrected for differences in temperature, assuming that the temperature coefficient is constant throughout the run, then for each value of the temperature coefficient the curve obtained has the same general shape as the original, but as the coefficient increases the minimum becomes shallower and moves to the left (see Fig. A). Only if we assume that the temperature coefficient remains practically constant at about 2

*Tests made in the laboratory of the C.S.I.R.O. Division of Building Research have shown that resistance between pools of mercury on a block of air-dry portland cement paste is between 1 and 2 megohms per in. separation of the mercury pools. Absolutely dry material would almost certainly have a much higher resistivity.

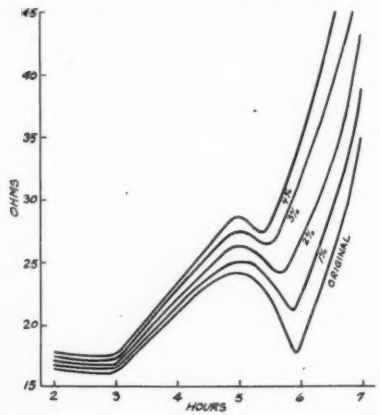

Fig. A—Curve 6 of Fig. 3 corrected with a series of constant temperature coefficients

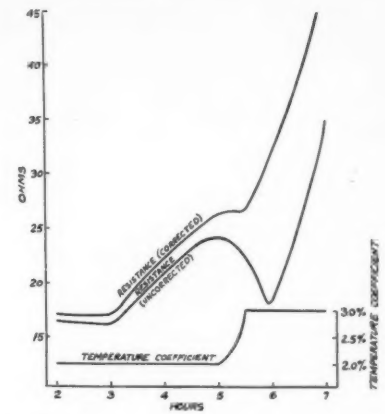

Fig. B—Curve 6 of Fig. 3 corrected with a varying temperature coefficient

percent up to 5 hr and then rises sharply to about 3 percent over the space of half an hour, does a horizontal point of inflection appear and the curve take on the form of curve 1, Fig. 5 (see Fig. B). The end of the horizontal step does not however coincide with the observed minimum shown in Fig. 5 and consequently, if Mr. Calleja's theories are correct, the minimum he observed is not the time of final set but merely a slightly distorted reflection of the temperature maximum.

In any case the "corrections" made to obtain the curve of Fig. B were extremely arbitrary and any small deviation from the selected values and rate of rise of the temperature coefficient will give a minimum in the curve again. Since such a steep rise in temperature coefficient can be due only to increasing concentration of soluble materials, it is implicitly assumed (in taking the end of the horizontal step as the end of the setting process) that the time of final set corresponds to a definite and fixed solution concentration, *i.e.*, a fixed free-water/cement ratio. In fact, the conditions necessary to give a curve with a horizontal step appear, when examined quantitatively, to be so restrictive and arbitrary that further experimental evidence is needed before the form of the isothermal curve can be settled.

From a practical point of view the effect of temperature corrections seems to be similar in either case, *i.e.*, "the position of the characteristic point indicating the . . . end of the setting process" has been moved to the left by between a quarter and half an hour for any value or arrangement of values of the temperature coefficient which is consistent with the known behavior of electrolytes. This increases the discrepancy observed by Mr. Calleja between setting time as indicated by resistance measurement and the maximum of the temperature-time curve. There are good and well established reasons for concluding that the maximum of the temperature-time curve represents closely the end of the reactions accompanying the setting process. Consequently any real difference between the maximum of this curve and the

minimum, or flat, of the *R-t* curve must (at least until we have a more detailed knowledge of the reasons for these changes in resistance) cast doubt on the claim that the position of the resistance minimum indicates the time of final set. It is difficult to determine from the data given in the paper how large this difference may be, because the high heat capacity of the temperature measuring equipment will have delayed observation of the temperature maximum.

It appears from these recalculated curves that although the *R-t* curve may be of the form shown in curve 1, Fig. 5, it is equally likely that the minimum may be independent of any temperature changes; but we have insufficient quantitative information about the changes in cement paste which produce these variations in resistance or of their relation to the temperature maximum to be certain of their exact significance.

The old established use of the temperature maximum to determine the time of final set is thus more soundly based (in our present state of knowledge) than that of the resistance minimum. If, however, reasons for the observed variations of electrical resistance can be determined in detail, they may well throw a new light on the setting mechanism.

ACCURACY, SENSITIVITY AND UNIFORMITY OF RESULTS

Despite the theoretical uncertainties regarding the significance of the resistance minimum, its observation as a means of determining the time of final set would be worth while if some practical advantage could be obtained over the older methods.

Although it is stated with respect to the resistance method that "the largest discrepancies registered . . . are within a maximum variation of 15 minutes" (p. 530), insufficient evidence is given of the number of tests made to enable any estimate to be made of the statistical worth of this statement, nor has any evidence been produced to show that a greater variation may be expected under the same experimental conditions with other methods of determining the time of set. The range of values for setting times by the method of temperature rise (taken from the curves of Fig. 3 and 4) is in each case less than the 15 minutes claimed for the resistance method. If the additional results for cement No. 2 given in Fig. 2 are considered, it will be seen that while the range of setting times as determined by temperature rise is increased to a total of 15 minutes those determined by resistance measurements are increased to 30 minutes.

Apparently changes in experimental conditions affect the results obtained by resistance measurement more than they do those obtained by more conventional temperature measurement. This seems to be a serious flaw in the resistance method if it is intended that it should be used as a standard method to determine characteristics of batches of cement.

If by accuracy and sensitivity the author means the precision with which setting time can be chosen, there appears to be little to pick between temperature and resistance methods. In Fig. 3 the peaks of the temperature

curves are sharper than those of the resistance curves, while in Fig. 4 the reverse is the case. In both methods it is equally difficult to pick the initial set (beginning of the first upward slope) with any precision.

Sharpness of the peak in the temperature-time curve can be largely controlled by suitable choice of thermal constants of the apparatus, but no such control is available for resistance measurement (except insofar as resistance is affected by temperature). The temperature method would therefore appear to have some advantage in this regard.

It has further been claimed in favor of the resistance method that it "makes it possible to record changes instantaneously. However, in recording temperatures this is not the case." Hence it is claimed that "the readings for the *R-t* curves are more precise than those of the *T-t* curves" and that the lag caused by the thermal capacity of the apparatus "fully accounts for the relative displacement of the two curves" (p. 535).

When the method of measuring temperature is considered, the lag in thermal measurements is only what would be expected; but why should the thermometric apparatus have such a large heat capacity? What is the necessity for placing the thermometer inside an iron tube of mercury? Why not mount it directly in the paste, after coating the bulb with a layer of soft paraffin to facilitate withdrawals? This method is in regular use in this laboratory for similar measurements on plaster of paris, and tests have shown that such greased thermometers are easily withdrawn from set cement paste (approximately 17 hr after mixing). If a suitable thermometer is not available the problem could be solved (and the heat capacity still further reduced) by employing an expendable thermocouple or couples. This would also give the thermometric method the other advantage claimed for resistance measurements, that of being adaptable to automatic recording (conclusion 6, p. 536).

While the high heat capacity of the apparatus used by Mr. Calleja must account for at least some of the relative displacement of resistance and temperature curves, it seems rash to claim that it fully accounts for the difference (p. 535) without more precise data on the actual magnitude of thermal lag of the instruments. Maximum temperatures reached and the apparent lag between temperature maximum and resistance minimum taken from curves for cements 1 and 2 are tabulated below.

	Max. temp. (nearest 1°C)	Apparent lag, min.	
Cement 1	44	10	Fig. 2
	44	15	Fig. 3 (curve 6)
	44	5	Fig. 3 (curve 7)
Cement 2	36	5	Fig. 2
	35	30	Fig. 4 (curve 8)
	36	35	Fig. 4 (curve 9)
	37	25	Fig. 4 (curve 10)

The variation in lag obtained with the same cement under supposedly identical experimental conditions, *e.g.*, 10 min. between curves 9 and 10 with cement 2 and between curves 6 and 7 with cement 1, and the range of up to 30 min. when the conditions are not identical (cement 2), suggest that further investigation is needed before it can be claimed that thermal lag fully accounts for the difference.

It appears therefore that although a more detailed investigation of the change of electrical resistance with time may help us to a better understanding of the mechanism of setting of hydraulic materials, our knowledge of such changes is not yet complete enough for us to say with certainty that they can advantageously be used to replace the older methods of measuring setting times.

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AUTHOR'S CLOSURE

Mr. Waters' attention, and also his doubts, are centered on two specific points: soundness of the theoretical basis of the proposed method and the accuracy, sensitivity, and uniformity of results compared to those of classical methods.

Referring to the theoretical basis of the method, Mr. Waters draws attention to curve 1, Fig. 5, which, it must be remembered, is not experimental, but hypothetical. Experimental data obtained to date on *R-t* isothermal curves are not considered final by this author.

Allowing for discrepancies between an experimental curve and a hypothetical curve taken as basis of an argument, curve 1, Fig. 5, can be correlated to experimental curves obtained by Shimizu[1] for isothermal conditions (provided care is taken to transform the conductivity-time data of Shimizu into resistance-time). Following Mr. Waters' argument, if the graphs obtained by Shimizu were to be confirmed in our experiments, then curve 1, Fig. 5, would be justified, and the minimum value shown on the *R-t* curves would be simply a reflection of the temperature rise.

Tests under isothermal conditions[2] (though the results are not final) seem to indicate that for a constant temperature of 25 ± 5 C, *R-t* graphs still exhibit a well defined minimum value at the end of setting. Should this be confirmed, Shimizu's results would require revision. It would also show, as stated by Mr. Waters, that curve 1, Fig. 5, has no real meaning as an *R-t* curve for isothermal conditions, and furthermore that the minimum point of the *R-t* curve would not depend, at least exclusively, on temperature changes.

In any case, it is most important to get reliable information on the changes in the cement paste which cause changes in its electrical resistance, and also on the minimum point in *R-t* isothermal curves. Our present state of knowl-

edge on setting does not provide this information. A wide variety of experimental data and their mutual relationships are essential before this problem can be well understood.

As regards the value of 1 percent per degree C as a temperature coefficient for electrical resistance of the paste, this is an assumed value. (But this factor has not been applied at all, for reasons given later.) Hence this coefficient is not an experimental datum, but is based on the following arguments.

The temperature coefficient of cement paste, after mixing, will probably have a value corresponding to that of solutions of bases and salts. It is reasonable to suppose that this value will not remain constant during setting but will vary. This variation will probably be such that the coefficient will decrease as setting progresses. This seems logical since ions will tend to lose mobility, among other reasons, because they will be increasingly subjected to the action of electrical fields of gel and colloidal micells, which emerge progressively as advocated in Michaelis' and other authors' theories.[3]

It would not be easy to determine, or even predict, the law of variation (reduction) against time of the temperature coefficient in the course of setting. But it is feasible to believe that, in accordance with the theories suggesting that ion movements decrease or even stop, the temperature coefficient, especially in the final stages of setting, would reach a substantially lower value than that corresponding to diluted ionic solutions. It is these reasons that have led the author to view as reasonable a mean temperature coefficient for electrical resistance of the paste lower than that corresponding to solutions cited above.

It is evident that the use of 1 percent per degree C as a correction factor would affect the shape of *R-t* curves and, what is more important, position of the minimum point in an almost negligible manner. This is clear from Fig. A, given by Mr. Waters. In any case, he admits it is difficult, if not impossible, to know which is the real value to be assigned to the temperature coefficient of the electrical resistance of the cement paste. Also, whatever the coefficient, the difficulty of applying it is even greater if it is remembered that it does not remain constant during the whole setting process, and that it would be even more difficult to find its law of variation.

But the adoption (more or less arbitrarily) of some coefficient, and the reasons for its selection (necessarily subjective to a large extent owing to lack of really definite data), seems a secondary matter. The main issue seems to be: Is it permissible and convenient to correct *R-t* curves by referring experimental electrical resistance data to a given temperature?

(a) The tests were made under almost adiabatic conditions as most nearly approaching actual conditions in practice. It would seem strange to correct data obtained under those circumstances to refer them to unreal isothermal conditions, since these have in no way been operative during the tests. Besides, the author agrees with Mr. Waters that to effect this correction it would be essential to know exactly both the temperature coefficient and its law of variation during setting. At the moment we are far from knowing this.

(b) In our conditions of work and in ordinary practice there is an increase of temperature during setting, and this influences the electrical resistance. The mere fact that this influence exists, which Mr. Waters recognizes in Fig. A, forces us to accept it. That is, the *R-t* curves must be accepted with the modifications caused by temperature variations, since these are inherent in the setting phenomenon.

(c) Isothermal *R-t* curves may exhibit a minimum, as provisional curves appear to show,[2] or their shape may be that of curve 1, Fig. 5. The minimum may simply reflect the temperature increase, it may be due to other causes, or, in the most general case, it may be due to both. But independent of its possible cause, it is an established fact that this minimum is closely related to the maximums on the *T-t* curves. On this fact rests most of the proposed method. So much so, that when this relationship does not exist the method cannot be applied, at least in the form initially proposed.[4]

Work has been conducted under almost adiabatic conditions, and a very near coincidence has been observed between the time coordinate of the *T-t* maxima and the *R-t* minima. On this, certain observations should be made.

(1) If the corrections suggested by Mr. Waters were applied to the *R-t* curves, the minima would be displaced to the left as shown in Fig. A so that the discrepancy with respect to the *T-t* maxima would be still larger.

(2) The corrections amount to a transformation of the almost adiabatic *R-t* curves into isothermals, *i.e.*, there would be a greater separation from perfect adiabatic conditions. Hence it seems reasonable that in perfect adiabatic conditions the deviation between *T-t* maxima and *R-t* minima would be much smaller than for an almost adiabatic state. Besides, to know the process of setting under isothermal conditions it is best for experimental conditions also to be isothermal. Then curve *R-t* would be directly obtained, without requiring any dubious correction. Shimizu worked that way,[1] and that is our present laboratory method.

(3) A much closer approximation between *T-t* maxima, and *R-t* minima is always possible if temperature of the paste is measured by means of thermocouples instead of thermometers.

Observations (a) and (b) show that it is not permissible to correct *R-t* curves by applying a doubtful temperature factor. Remarks (c) (1) and (2) also indicate that it is not advisable. In the work covered by the paper, temperature variation during setting has been studied only as a check on the *R-t* curves.

If the technique described is to be applied in practice, no temperature measurements have to be made, for otherwise the method is greatly complicated and consequently loses one of its greatest advantages. If no record of temperature is made, it is not possible to make a corresponding correction. Besides not being useful or convenient to correct *R-t* curves, it only adds unnecessary labor since if both measurements are made either the *R-t* curves or the *T-t* curves are superfluous.

Observations (c) (2) and (3) show moreover that there is close agreement between *T-t* maxima and *R-t* minima and that it is possible to obtain a time coordinate coincidence of these maximum and minimum points. Hence, for given experimental conditions, the minimum point of the *R-t* curve is closely related to the end of setting, since the *T-t* maximum is so related. Furthermore, this implies that if *T-t* curves have a sound theoretical basis,

this will also apply to *R-t* curves. Although with our present knowledge this is not well understood, it does not detract from the argument. This theoretical basis cannot be known until a thorough knowledge is gained, not only of the relationship between *R* and *T*, but also of that linking *R* with the many variables which could and should be taken into account in connection with the setting phenomenon.

The author also disagrees with the suggestion that the proposed method is a cumbersome way of performing the old established technique of determining the setting time by observing the temperature rise of paste. Measurement of a temperature is always, at least under our conditions of work, as troublesome as the measurement of an electrical resistance, and usually much more so. Two examples will make this clear.

(1) Sometimes measurements have to be made at a working site, for example, close to a dam under construction. In such a case it is evident that measurement of a temperature is far more inconvenient than measuring a resistance or conductivity, assuming in each case adequate equipment.

(2) The same would occur if the process of setting and hardening were to be observed at plants making prefabricated units kept inside a curing chamber or autoclave. It is much easier to measure from outside the electrical resistance of a mass of mortar or concrete than a temperature. Besides, measurement of temperature would give a value at a given point of the mass for each thermocouple, while the electrical resistance would be given by a value relating to the whole mass contained between the electrodes. This in our opinion gives a more representative result.

The number of tests to which the expression "the largest discrepancies registered . . . are within a maximum variation of 15 minutes" (p. 530) refers, was 38. Curves shown in the original paper were selected at random from among the 38 results. Altogether more than 200 tests have been done now with the new method. In almost all these it has been observed that peaks of resistance curves are sharper than those of temperature curves. This makes it unnecessary to look for doubtful methods to increase the sharpness of these peaks, and reduces the advantage which Mr. Waters attributes to the temperature method.

Although the temperature and resistance methods are equally convenient if automatic recording is adopted (provided temperature is recorded by thermocouples) nonetheless, the foregoing examples, among others, show that the advantage lies once again on the side of the new method.

The author does not believe that in the new method variations in experimental conditions should affect the results more than in the case of thermal methods. But even so, it is clear that if a standardized procedure were to be developed, careful thought would be given as to most suitable experimental conditions.

Summarizing, we have the following points:

(1) There should be no concern about correcting, by means of a temperature coefficient, the *R-t* curves obtained adiabatically.

(2) The existence of a sound theoretical basis for this method should not

be denied simply because for the moment it is not well known. It is believed that such a basis exists.

(3) The proposed method has all the advantages of thermal methods. Conversely, it lacks its disadvantages.

Finally, importance is attached to any contribution tending to clarify the theoretical significance of the *R-t* curves. This would lead to a better and more complete knowledge of the setting phenomenon.

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Title No. 48-36

Calcium Chloride in Concrete*

By J. J. SHIDELER†

SYNOPSIS

Calcium chloride has been used for many years to accelerate the early strength development of concrete. Its effects on other properties of concrete are not as well known. This paper presents test results on the effects of type of cement, age, temperature and mix proportions on the strength of concrete containing $CaCl_2$, as well as the effect of $CaCl_2$ on the volume change, heat generation, time of set, alkali-aggregate reaction, resistance to abrasion and sulfate attack of concrete. References to the work of other investigators and some of their general conclusions are included.

INTRODUCTION

Numerous tests have been performed by the Bureau of Reclamation to determine the effect of $CaCl_2$ on various properties of concrete. $CaCl_2$ accelerates early strength development and is particularly useful when concrete is placed during freezing weather. One percent $CaCl_2$, by weight of cement, along with air entrainment, has been specified by the Bureau for concrete subject to freezing conditions at early ages. However, $CaCl_2$ generally increases volume changes of concrete either under moist curing or drying conditions. It also increases the alkali-aggregate reaction and reduces the resistance of concrete to sulfate attack.

EFFECTS ON STRENGTH

The principal reason for using $CaCl_2$ in concrete is to produce early strength, required to permit early removal of forms or to provide protection against freezing. The effectiveness of $CaCl_2$ in promoting early strength is dependent upon many factors, including amount of $CaCl_2$, type and brand of cement, richness of mix, temperature of concrete, and curing conditions. In all tests discussed $CaCl_2$ was used as an integral addition to the concrete and not as a surface application.

Type of cement

$CaCl_2$ in the amount of 1 or 2 percent by weight of cement increases the early strength of all types of cement. Typical results are shown in Fig. 1. These data were obtained in the laboratories of the Bureau of Reclamation,[1]

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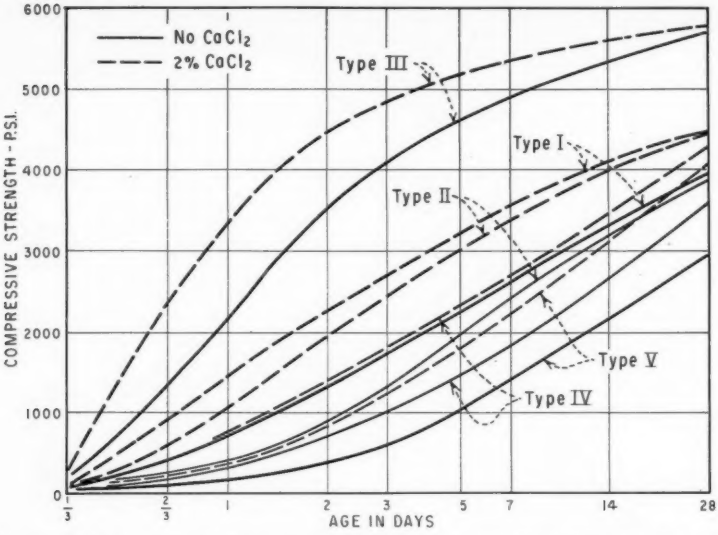

Fig. 1—$CaCl_2$ increases the compressive strength of concretes made with all types of cement

but numerous investigators have reported similar results. Cylinders were made from similar mixes containing each of the five types of cement, with a 0.55 water-cement ratio, a cement content of about 5.8 sacks of cement per cu yd, either with or without an addition of 2 percent $CaCl_2$. The cylinders were cast at 70 F, and cured at 100 percent relative humidity and 70 F. This study shows that each of the concretes was considerably improved in early strength development by the addition of the salt. In each of these concretes the $CaCl_2$ developed strengths in less than 1½ days equal to the 3-day strength of concrete without $CaCl_2$.

The 28-day strengths for the plain and admixed concrete made of Type III cement were approximately equal, but the strength ratio of plain and admixed concrete made of other cement types favored the $CaCl_2$ concretes to a considerable extent.

Table 1 gives the length of time required for each of these concretes to obtain a strength of 2000 psi.

TABLE 1—TIME REQUIRED FOR VARIOUS CONCRETES TO ATTAIN 2000 PSI

Cement type	Time, days	
	Plain concrete	Admixed concrete
I (Normal)	4	1.7
II (Modified)	5	2
III (High early strength)	1	0.6
IV (Low heat)	9	4
V (Sulfate resisting)	12	6

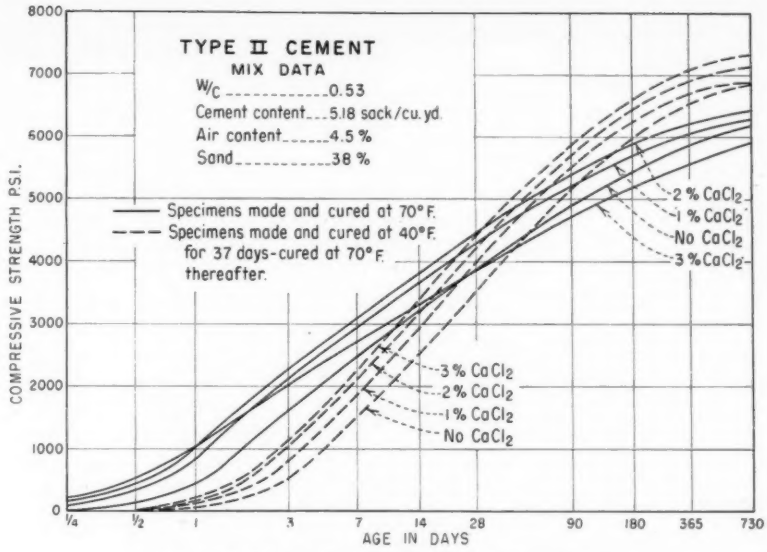

Fig. 2—$CaCl_2$ may be detrimental at higher temperatures

In another series of tests made primarily to study the combined effect of an air-entraining agent and $CaCl_2$, compressive strengths were obtained to age 2 years. Some of the 6 x 12-in. cylinders were cast and maintained at 70 F, other were cast and maintained at 40 F for 37 days prior to storage at 70 F. All specimens were continuously moist cured. $CaCl_2$ increased the strength at all ages, with one exception. Specimens cast at 70 F had higher early strength but lower strength after 6 months than specimens cast at 40 F (Fig. 2). For these 40 F tests, each 1 percent addition of $CaCl_2$ increased the strength about 200 psi for ages between 3 days and 2 years. At 70 F, strength differences slightly greater than this were obtained at early ages. At 70 F, 3 percent $CaCl_2$ produced the highest strength at 1 day, but produced the lowest strengths after 28 days. This indicates that as far as strength is concerned more $CaCl_2$ can be used safely at low temperatures than at higher temperatures. The greatest percent strength increase due to $CaCl_2$ was produced at early ages and with the lower temperatures.

Although a precipitate was formed when the $CaCl_2$ and the air-entraining agent were added to the mixing water, the effectiveness of neither admixture was impaired to any noticeable extent by the combination.

Although the primary purpose of using $CaCl_2$ is to produce high early strengths, greater strengths may persist for several years. At Grand Coulee Dam,[3] 6 x 12-in. concrete cylinders made with Types I, II, and V cement with cement contents of 4½, 6, and 7½ sacks per cu yd and 0 and 2 percent $CaCl_2$ were stored in clear water in an outdoor pond for 5 years. The con-

crete with $CaCl_2$ produced equal or greater strength at all ages for all combinations. Fig. 3 shows the strength obtained using a Type II cement; similar curves were obtained for the other two cements.

Abrams[4] presented data on the compressive strength of concrete and mortar, made of four brands of cement and having admixtures of from 0 to 10 percent $CaCl_2$. These data show that in the vast majority of tests, concrete containing the salt at concentrations not exceeding 4 percent had greater strength than untreated concrete at test periods ranging from 2 days to 3 years; but 7 percent and higher percentages of calcium chloride produced lower strengths at all ages.

Russell[5] found that 2 percent $CaCl_2$ was beneficial in promoting higher early strengths in modified cubes made with standard and high early strength cements when placing temperatures were 40, 50, 60 and 70 F. At the end of a year the majority of cubes containing the salt continued to show higher strength than the plain mortar cubes. Four and 5 percent $CaCl_2$ increased early strength but reduced strengths at 1 year. With a high early strength cement, 5 percent $CaCl_2$ was detrimental at all ages.

Temperature

The temperature at which a concrete mass is placed and cured is an important factor in controlling the rate at which concrete develops strength.

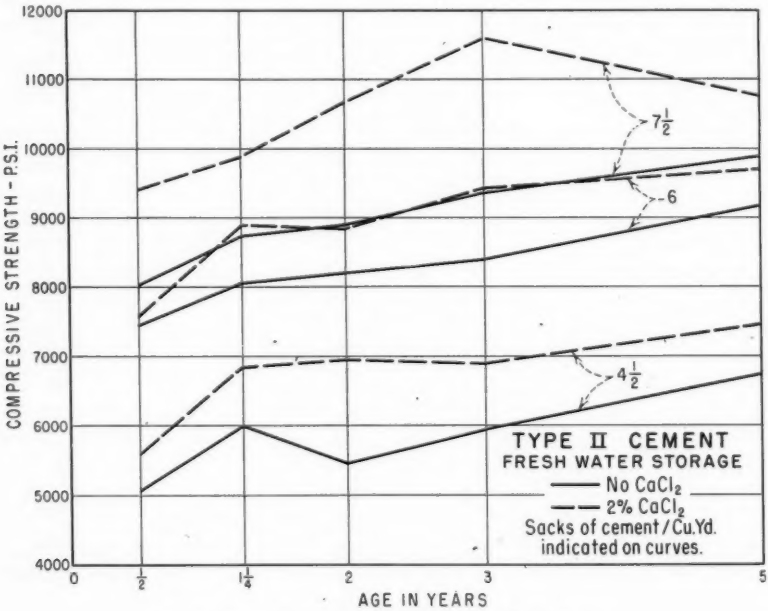

Fig. 3—$CaCl_2$ increases compressive strength at all ages

$CaCl_2$, because of its influence in promoting accelerated hydration of cements, compensates for the normally slow strength development of concrete placed during cold weather. The Bureau of Reclamation laboratory[6] has presented data for low temperature placement of concrete containing 0, 1, and 2 percent $CaCl_2$, with Types II and V cements and a 0.50 water-cement ratio. In these tests, concrete with an initial temperature of 50 F was cured for 3 days at that temperature prior to storage at freezing or near freezing temperatures. Fig. 4 and 5 show that concrete containing 2 percent $CaCl_2$, either stored at 32 F after 3 days at 50 F, or frozen for 3 days after 3 days at 50 F, had greater strength at all ages than concrete without $CaCl_2$ stored continuously at 50 F. Concrete containing 1 percent $CaCl_2$ showed an advantage similar to that containing 2 percent of the salt but to a lesser extent. Fig. 6 shows the strengths of concrete made with Type V cement and 0 and 2 percent $CaCl_2$ additions, given one daily cycle of freezing at 10 F and thawing at 50 F in air for 25 days. Similar curves were obtained with a Type II cement.

Additional data were also obtained on concrete containing 0 and 1 percent $CaCl_2$, mixed and maintained at 50 F for 24 hours prior to storage at temperatures of 16, 50, 70 and 100 F for periods to 1 year (Fig. 7). These 6 x 12-in. cylinders contained Type II cement with 4 percent air and were sealed in metal containers. At 16 F the concrete with $CaCl_2$ at all ages had approximately twice the strength of concrete with no $CaCl_2$ and a strength of 2600 psi at 1 year. Frozen specimens were thawed in water prior to testing. $CaCl_2$ and higher temperatures develop higher strength at early ages, but all specimens, regardless of storage temperature or percentage of $CaCl_2$, with the exception of the 16 F specimens, had approximately the same strengths at 1 year. Tests conducted at 33 F were discontinued at 28 days due to the difficulty of maintaining this particular temperature, but during this period the concrete containing $CaCl_2$ showed considerably higher strengths.

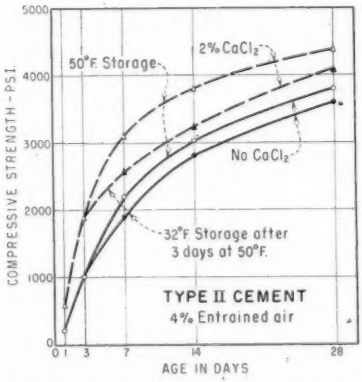

Fig. 4—$CaCl_2$ increases strength at low temperatures

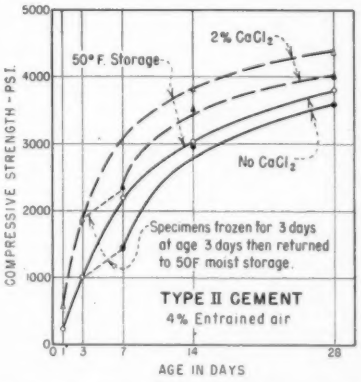

Fig. 5—$CaCl_2$ provides cold weather protection

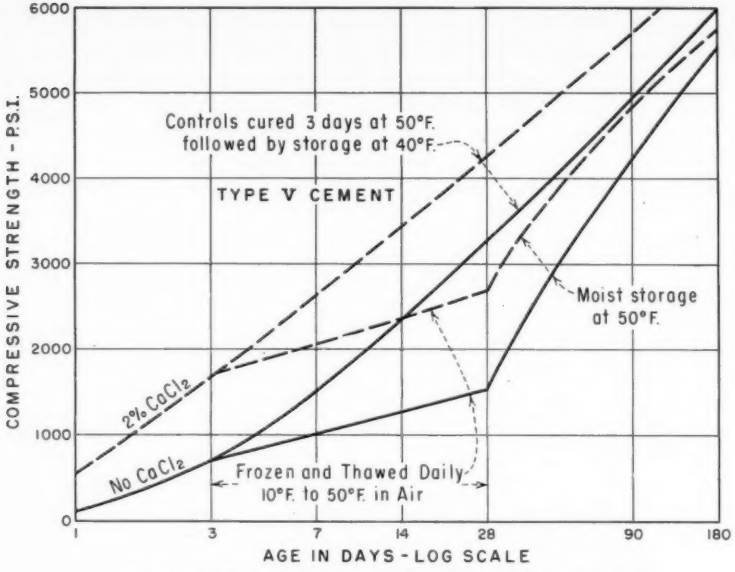

Fig. 6—$CaCl_2$ increases strength of concrete frozen at early ages

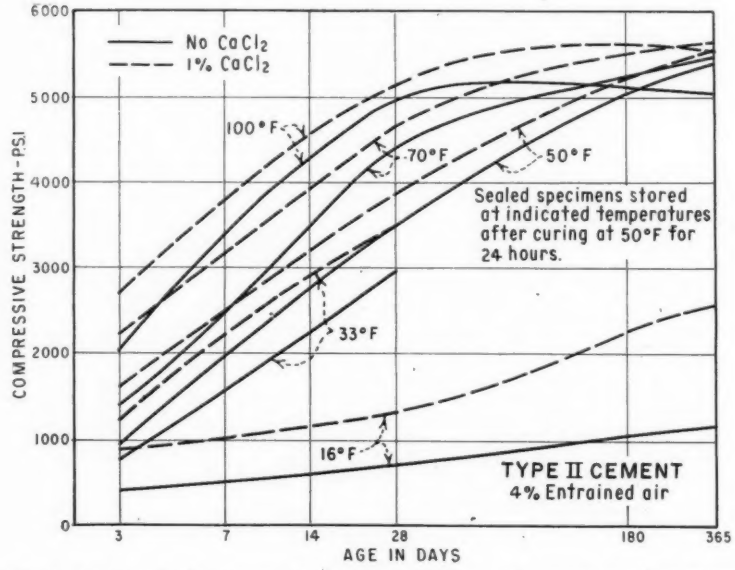

Fig. 7—1 percent $CaCl_2$ increases the compressive strength of concrete stored at various temperatures

Tests conducted by the National Bureau of Standards[7] in which 0, 2, 3 and 4 percent $CaCl_2$ were used in concretes mixed at 70 F and immediately placed and cured at 20, 25, 32 and 40 F show that the $CaCl_2$ increased the strength of the concrete considerably at all ages from 1 to 28 days. These data show the advantage of using $CaCl_2$ when the concrete may be subjected to freezing temperatures shortly after placement. Although the higher percentages of $CaCl_2$ produced higher strength at all temperatures and ages, 4 percent $CaCl_2$ produced only slightly higher strengths than 2 percent.

Clemmer[8] conducted an extensive series of tests of concrete placed and cured at temperatures ranging from 25 to 70 F, and has shown the beneficial effect of $CaCl_2$ at low temperatures.

Cement content

Bureau tests have shown that $CaCl_2$ is more effective in increasing the early strength of rich concrete than of lean concrete. Fig. 8 shows the effect of 1 percent $CaCl_2$ on the strength of concrete containing five, six and seven sacks of Types II and V cements. At 1 day, 1 percent $CaCl_2$ produced a strength gain equivalent to the addition of 2 sacks of cement and at 3 days more than 1¼ sacks. At later ages an additional sack of cement produced considerably more strength gain than the 1 percent $CaCl_2$. Fig. 9 presents the data for the Type V cement in a different manner and shows that at 28 days each additional sack of cement increased the strength about 1000 psi and that 1 percent $CaCl_2$ increased the strength only about half this amount. Curves for the Type II cement were similar and showed that 1 percent $CaCl_2$ produced a greater increase in strength at early ages than an additional sack of cement, but after 7 days the additional cement produced the greater

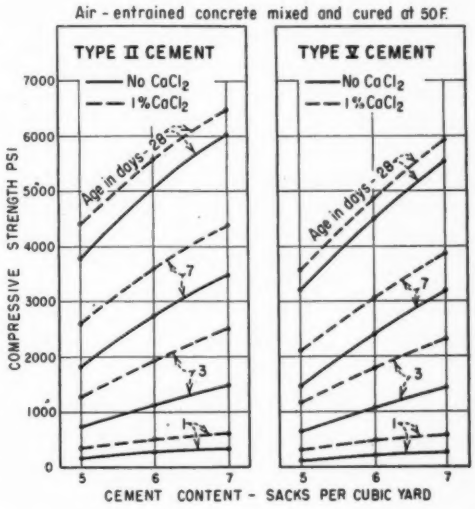

Fig. 8—$CaCl_2$ increases the strength of concrete of different cement contents and at different ages

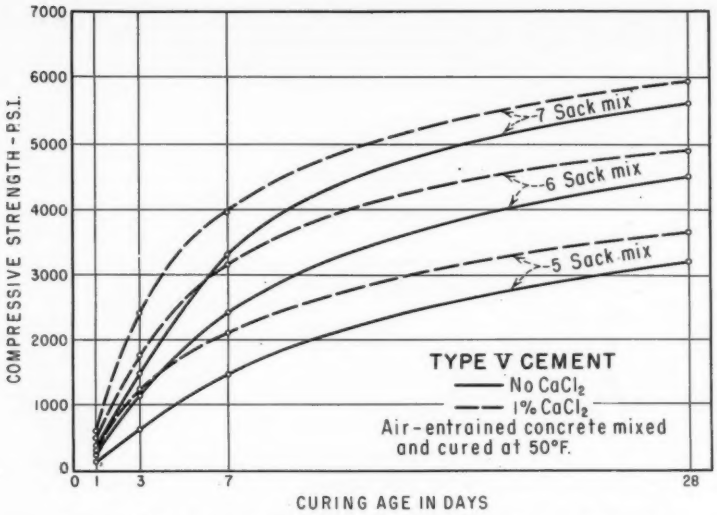

Fig. 9—$CaCl_2$ increases the strength of concrete made with Type V cement

strength. Where Type V cement is required to provide resistance to sulfate attack, the additional cement should be used instead of $CaCl_2$ as discussed later.

Clemmer[8] has presented similar data using 2 percent $CaCl_2$ and standard (Type I) and high-early (Type III) cements. At 1 day, 2 percent $CaCl_2$ produced a strength gain greater than that obtained with 2 additional sacks of cement, and at 7 days equal to about an additional ½ sack of cement. His data also show that $CaCl_2$ produces greater unit-strength gain in rich concrete than in lean concrete, but that $CaCl_2$ generally produces a larger percent strength gain in the leaner mixes.

Abrams[4] conducted an extensive study including 0 to 10 percent $CaCl_2$ with mix proportions ranging from 1:2 to 1:7 by volume. He concluded that the increase in strength due to the addition of 2 percent $CaCl_2$ was practically negligible for a 1:7 mix, it increased with the cement content up to about a 1:4 mix, and for mixes richer than 1:4 the strength increase remained about the same. He also concluded that no advantage was gained by using percentages of $CaCl_2$ greater than 2 or 3 percent of the weight of cement regardless of mix proportions.

Flexural strength

Most investigators show that $CaCl_2$ increases the flexural strength during the first few days but has little effect at later ages. In one series of tests performed by the Bureau,[15] concrete prisms, with 0, 1, 2 and 3 percent $CaCl_2$, were fog cured for 14 and 28 days, then dried at 70 F and 50 percent relative

humidity for 18 months, then tested in flexure. For specimens fog cured 14 days, 2 percent $CaCl_2$ decreased the flexural strength about 10 percent, and for specimens fog cured 28 days, increased the flexural strength about 10 percent.

Russell,[5] conducted a study of flexural strength of concrete cured at 40, 50, 60 and 70 F with Types I and III cements and 0, 2 and 4 percent $CaCl_2$. Flexural strengths were obtained only at 1, 2 and 3 days' age. His data show that $CaCl_2$ increased the early flexural strength. At 60 and 70 F there was little difference between 2 and 4 percent $CaCl_2$, but at 40 F, 4 percent $CaCl_2$ produced considerably higher strengths than 2 percent $CaCl_2$. The Type III cement was used at 40 F only and 4 percent $CaCl_2$ showed no advantage over 2 percent.

Vollmer[10] obtained flexural strength data on concretes cured nine different ways, and shows that $CaCl_2$ increased the flexural strength at early ages but decreased it slightly at 28 days. The decrease at later ages may have been due to the difference in curing conditions, as the concrete without $CaCl_2$ was moist cured 72 hours and the concrete with $CaCl_2$ only 18 hours. Clemmer and Burggraf[24] have shown that $CaCl_2$ increased the 28-day transverse strength of concrete prisms cured at temperatures below freezing.

Stanton[25] has presented data which show that $CaCl_2$ had no effect on the flexural strength of beams taken from a concrete highway at ages of 28 days and 6 months. Companion compressive strength specimens showed no strength gain due to $CaCl_2$ at these ages, but showed a 26 percent increase at 11 years' age.

Olson,[27] Tennessee Department of Highways, showed that 1 and 2 percent $CaCl_2$ was detrimental to flexural strength at ages of 7 days to 3 months. Other investigators have shown that $CaCl_2$ increased the flexural strength to 7 days but had either no effect or slightly decreased the flexural strength at 28 days and 1 year. Reliable data show that 2 and 4 percent $CaCl_2$ decreased the 28-day flexural strength of moist-cured concrete 10 and 20 percent, respectively, but had no effect on concrete dried after 7 days.

Tensile strength

Platzman[11] has shown that $CaCl_2$ in amounts of about 0.5, 1.0 and 1.5 percent had no effect at 7 and 28 days' age and was slightly detrimental at age 2 and 3 months. Levens[12] showed that from 2 to 5 percent $CaCl_2$ increased the 28-day tensile strength of mortar specimens dried after 1 day, but that 2 percent of the salt produced the highest tensile strengths. Under moist curing, more than 2 percent $CaCl_2$ produced lower strengths than specimens without the salt.

OTHER PROPERTIES

Heat of hydration

Several investigators have determined that $CaCl_2$ increases the heat of hydration of cement during the first few hours, but has little effect after 1 day. In tests conducted by the Bureau of Reclamation on Type II cement,

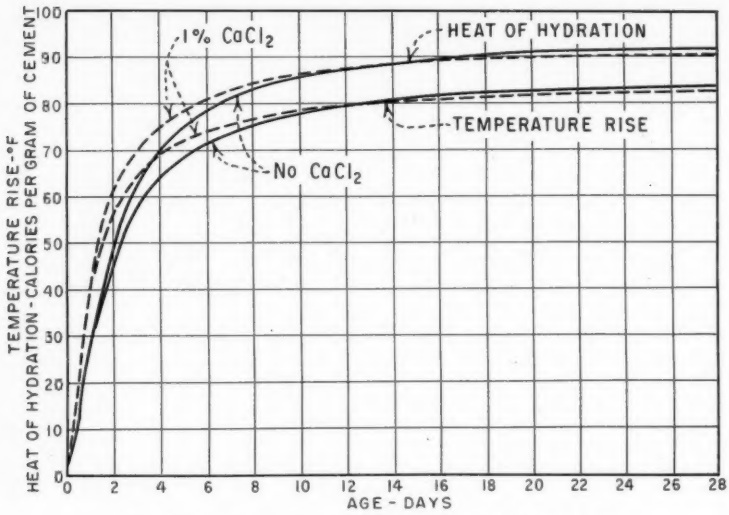

Fig. 10—$CaCl_2$ increases heat development only at early ages

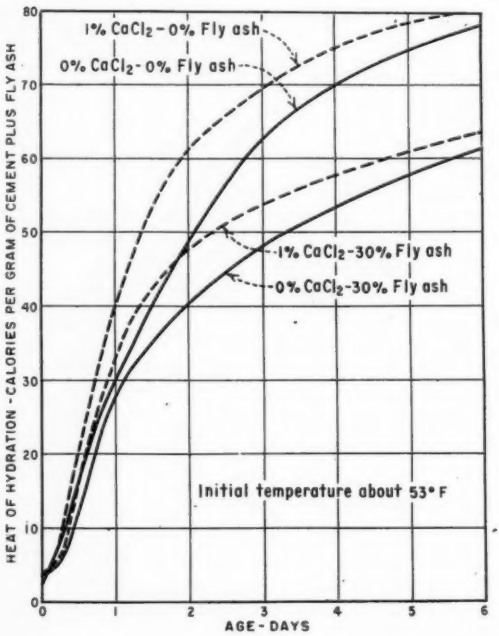
1% $CaCl_2$ - 0% Fly ash
0% $CaCl_2$ - 0% Fly ash
1% $CaCl_2$ - 30% Fly ash
0% $CaCl_2$ - 30% Fly ash
Initial temperature about 53° F
HEAT OF HYDRATION - CALORIES PER GRAM OF CEMENT PLUS FLY ASH
AGE - DAYS

Fig. 11—$CaCl_2$ increases early heat development

the addition of 1 percent $CaCl_2$ increased the 1-day heat of hydration 30 percent. This includes the effect of increased rate of heat generation caused by the higher temperatures obtained with $CaCl_2$. The temperature rise was determined over a 28-day period in an adiabatic calorimeter room on a 700 lb mass-concrete specimen. Tests included concrete specimens containing 5.3 sacks of cement per cu yd, a 0.53 water-cement ratio, with and without $CaCl_2$ and with and without a 30 percent fly ash replacement of cement. The heat of hydration was computed from the temperature rise data and the average specific heat determined for the concretes. The initial temperature for these tests was about 53 F. Test values are shown graphically in Fig. 10 and 11. Fig. 11 is an expanded graph of the first few days and shows more clearly the effect of $CaCl_2$ on the temperature rise during early ages. Compressive strength curves for these concretes are included. The addition of $CaCl_2$ resulted in a greater rate of hydration for the first 36 hours of the test, as indicated by Fig. 12. When the increased rate of hydration, due to the higher temperature, is considered, it is concluded that the $CaCl_2$ is active only during the initial 24 hours.

$CaCl_2$ increases the rate of temperature rise and causes the maximum to occur earlier (Fig. 12). With this particular cement the maximum rate occurred at about 12 hours without $CaCl_2$ and at 8 hours with $CaCl_2$.

Bureau of Reclamation tests on concrete containing fly ash indicate the increase in temperature rise to be no greater than would result from the cement and $CaCl_2$ alone, leading to the conclusion that $CaCl_2$ is not effective in increasing the rate of heat generation of fly ash.

Tests conducted by Paul Rapp[13] showed similar relationships in the heat of hydration with a much smaller difference due to $CaCl_2$ at 24 hours. A

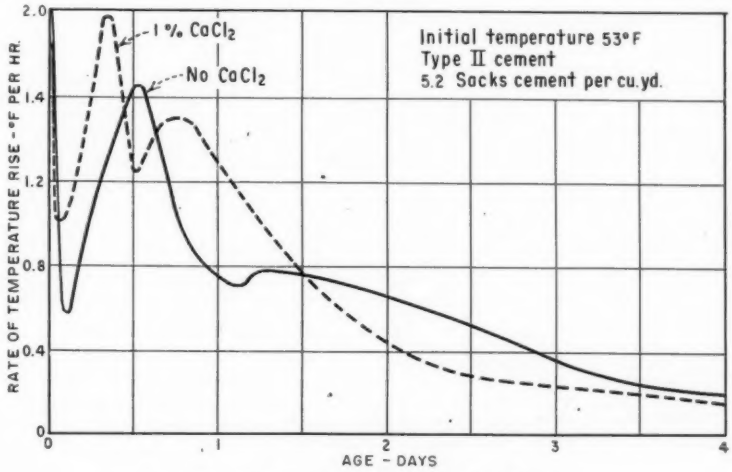

Fig. 12—$CaCl_2$ accelerates early heat development

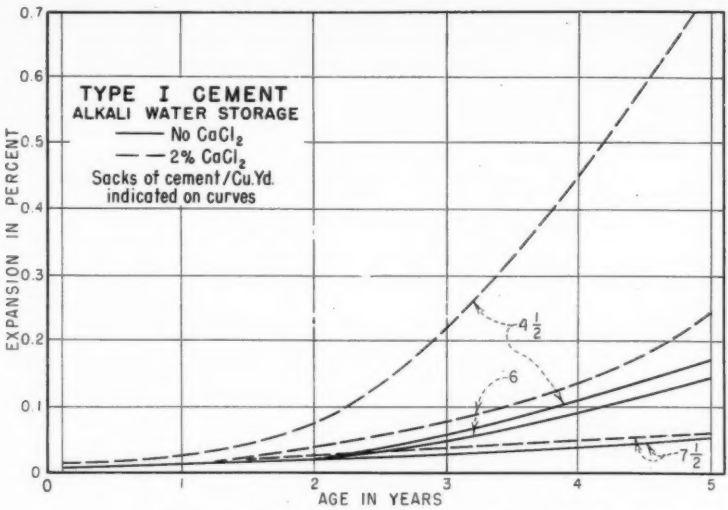

Fig. 13—$CaCl_2$ decreases resistance to sulfate attack

small neat cement sample enclosed in a thermos bottle was used for these tests. The effect of temperature rise was eliminated to a large extent, which may account for a portion of the difference between these tests and the adiabatic tests conducted on concrete by the Bureau of Reclamation.

Sulfate resistance

On the Grand Coulee Project[3] sulfate attack on concrete has imposed a severe problem and has been under study for several years. A very extensive series of tests, including air-entraining agents and $CaCl_2$, has been in progress. Types I, II, and V cements, with cement contents of 4½, 6 and 7½ sacks per cu yd, with 0 and 2 percent $CaCl_2$ were used. Prisms and cylinders were stored in fresh water and in water of high sulfate content in outdoor ponds. These specimens were tested for length change, compressive strength, and dynamic modulus, and 5-year data are now available.

$CaCl_2$ contributed to lower resistance to sulfate attack in all cases. Fig. 13 shows the expansion of 3 x 3 x 16¼-in. prisms containing a Type I cement and stored in the sulfate pond. The 4½-sack mix with 2 percent $CaCl_2$ showed 0.73 percent expansion at 5 years' age, while the prisms without $CaCl_2$ showed only 0.17 percent. The 6-sack mix showed expansion of 0.28 and 0.14 percent with and without $CaCl_2$, respectively. The 7½-sack mix had an expansion of about 0.05 and this was only slightly increased by $CaCl_2$. The prisms containing Type II cement showed considerably greater expansions than those with Type I cement, and in the leaner mixes the $CaCl_2$ was particularly detrimental (Fig. 14). Both the Type I and II cements had a

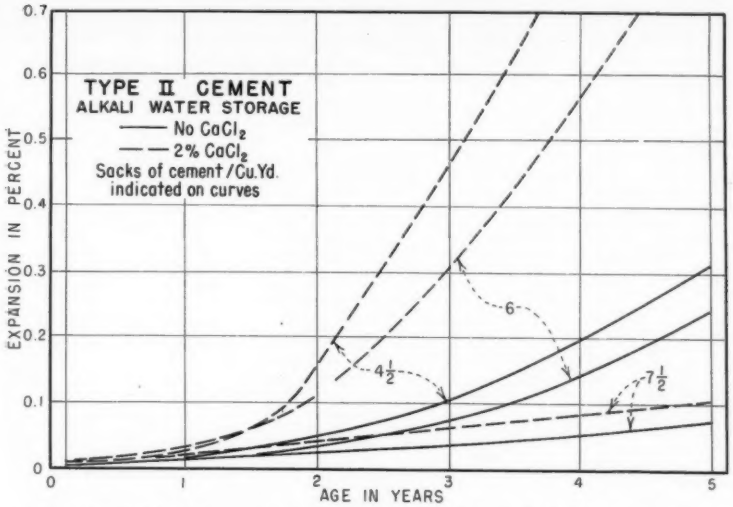

Fig. 14—$CaCl_2$ decreases resistance to sulfate attack particularly in lean concrete

low C_3A content, about 5 percent, which probably accounts for their similar performance.

The PCA Long Time Study of Cement Performance in Concrete[26] has shown that Type I cements are generally less resistant to sulfate attack than

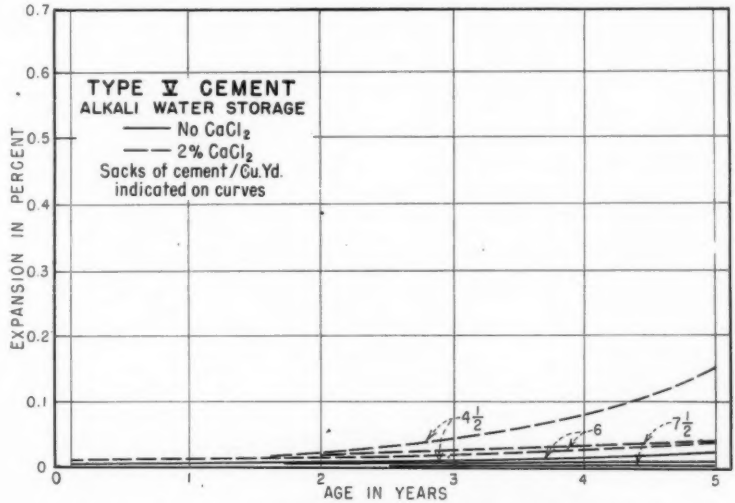

Fig. 15—$CaCl_2$ least detrimental to sulfate resistance with Type V cement

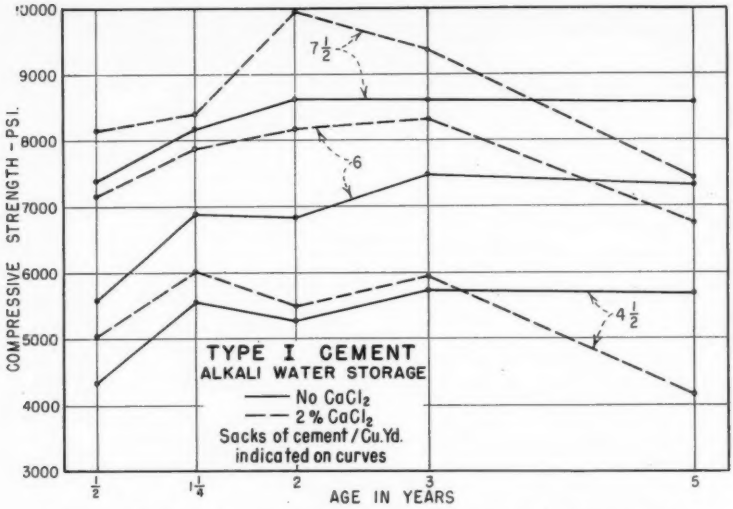

Fig. 16—$CaCl_2$ reduces compressive strength of concrete under sulfate attack

Type II cements. The Type V cement was very effective in limiting the expansion (Fig. 15), but prisms containing $CaCl_2$ expanded more than those without the admixture.

Compressive strength results show trends similar to those indicated by length change measurements of the prisms. That is, $CaCl_2$ was generally detrimental to concrete exposed to sulfate attack, particularly the concrete of low cement content. The concretes containing 7½ sacks of cement with a Type II or V cement were the only ones not adversely affected by $CaCl_2$. Compressive strength of 6 x 12-in. concrete cylinders, containing a Type I cement, are shown in Fig. 16. These specimens were stored in the sulfate pond. The $CaCl_2$ produced higher strengths for 3 years but considerably lower strengths at 5 years' age. The concretes with the lower cement contents of Type II cement and containing $CaCl_2$ were severely damaged by sulfate attack but at 5 years the $CaCl_2$ was not detrimental in the 7½-sack concrete (Fig. 17). Type V cement showed greater resistance to attack, but only the 7½-sack mix was not adversely affected by $CaCl_2$ at age 5 years (Fig. 18). The use of air-entrainment counteracted to a large extent the detrimental effect of $CaCl_2$.

Alkali-aggregate reaction

$CaCl_2$ generally increases expansion caused by alkali-aggregate reaction. Certain aggregates react with the alkalies in cement to produce a silica gel which by swelling may produce stresses great enough to rupture the concrete. However, when this reaction is effectively controlled by the use of both low-alkali cement and pozzolan, the effect of the $CaCl_2$ appears to be unimportant.

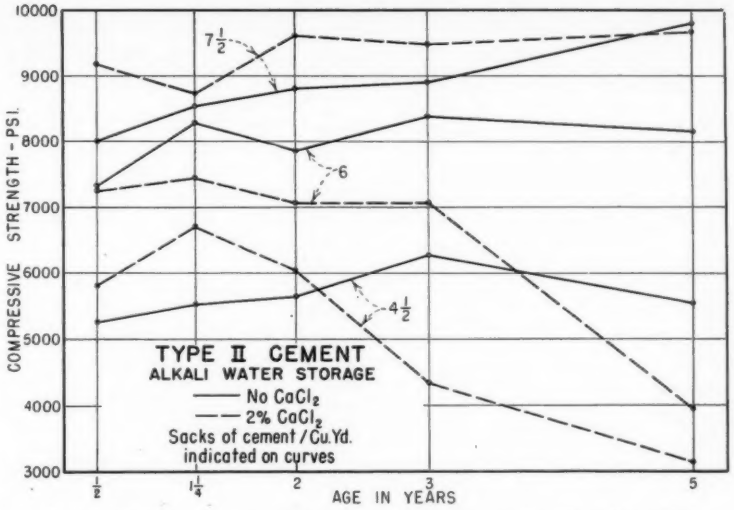

Fig. 17—$CaCl_2$ reduces compressive strength of lean concrete under sulfate attack

In the standard mortar-bar test, a high-alkali cement, 1.16 percent Na_2O and 0.04 percent K_2O, was used with three reactive sands, Republican River, Kimball, and Pyrex glass, and 0, 1 and 2 percent $CaCl_2$. Bars were stored

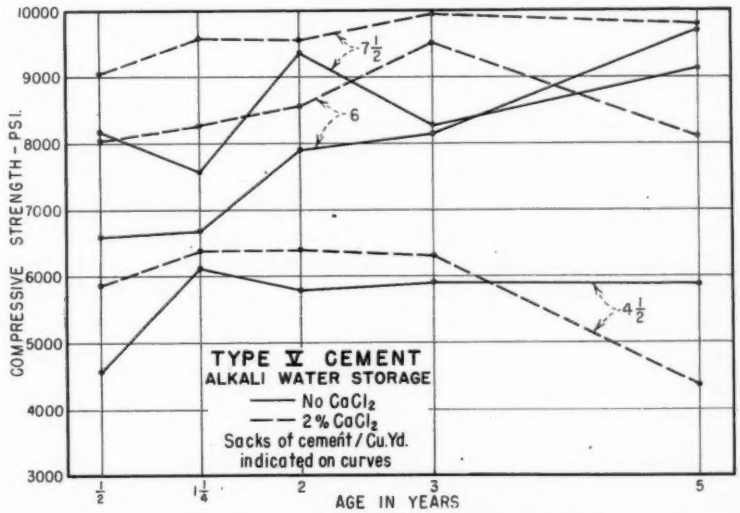

Fig. 18—$CaCl_2$ has little effect on sulfate resistance with Type V cement

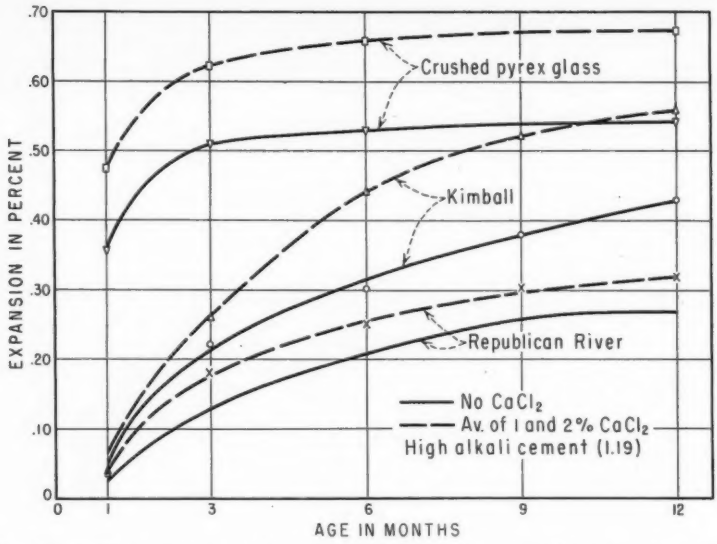

Fig. 19—$CaCl_2$ increases expansion caused by high alkali cement with reactive aggregates

at 100 F in sealed containers with a little water in the bottom of the containers but not in contact with the bars. $CaCl_2$ increased the expansion about 30 percent at age 1 year for each of these aggregates. These expansions were in the range of 0.27 to 0.70 percent as shown in Fig. 19. When low-alkali cement, 0.11 percent Na_2O and 0.57 percent K_2O was used, all expansions were reduced to the range of 0.08 to 0.16 percent. For two aggregates with the low-alkali cement, bars containing $CaCl_2$ showed 50 to 100 percent more expansion than the bars without $CaCl_2$, while in bars containing the third aggregate the expansion was reduced by the addition of $CaCl_2$.

In another series of tests, now in the preliminary stage, an attempt is being made to further evaluate the effect of $CaCl_2$ on the alkali-aggregate reaction. High- and low-alkali cements are used with Republican River and Pyrex glass aggregates. Each of three pozzolans and 0 to 4 percent $CaCl_2$ are also included with each cement-aggregate combination. Mortar bar specimens have been under test only 3 months, but some trends are apparent. Fig. 20 indicates that $CaCl_2$ is detrimental with Republican River aggregate and is slightly beneficial with the Pyrex glass aggregate.

Freezing and thawing

Many tests have been made by the Bureau of Reclamation to determine the effect of $CaCl_2$ on the resistance of concrete to freezing and thawing. All early tests were made on non-air-entraining concrete and generally these tests indicated that concrete containing not more than 2 percent $CaCl_2$ gave

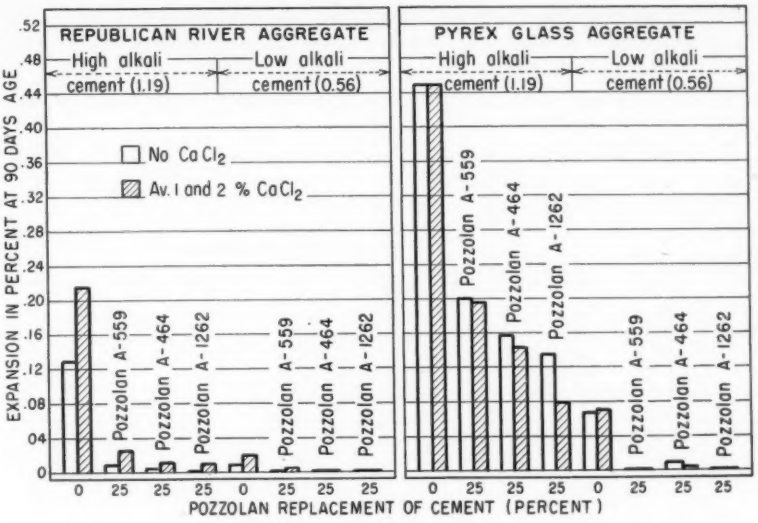

Fig. 20 (Above)—$CaCl_2$ not detrimental when pozzolan and low-alkali cement control alkali reaction

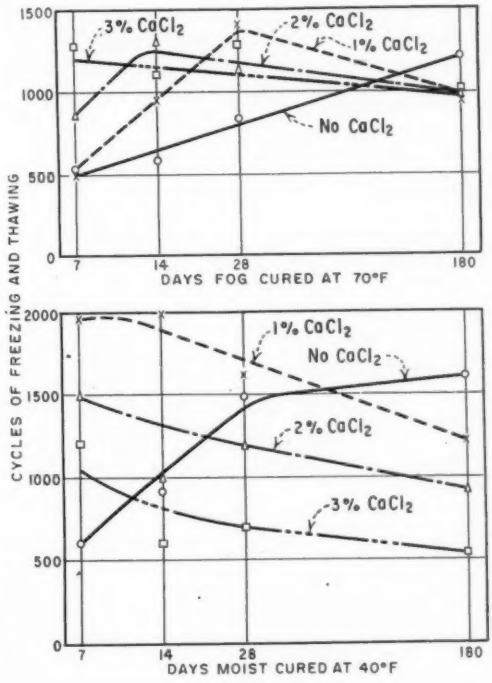

Fig. 21—$CaCl_2$ improves durability of air-entraining concrete at early ages, but is detrimental at later ages

higher resistance to freezing and thawing than concrete without the accelerator. However, these concretes were immature when subjected to freezing and thawing, usually within 28 days after placing.

Since that time, many air-entraining concretes have been tested at many ages and under different curing conditions. While these tests also indicate that $CaCl_2$ improves the durability of concretes tested at early ages, they almost invariably show that $CaCl_2$ decreases durability at later ages.

This latter statement is demonstrated in Fig. 21, which shows the effect of $CaCl_2$, curing temperature, and period of moist curing on the durability of air-entraining concrete. A greater difference in durability was obtained for the concretes moist cured at 40 F than at 70 F. Both graphs, however, show the same trend, higher durability at early ages for the concretes containing $CaCl_2$ and lower durability at later ages. The graph for the concretes cured at 70 F shows that concrete containing 3 percent $CaCl_2$ is the most durable when tests are started after the concrete was cured only 7 days, but that durability was decreased after additional curing. The optimum durability at 40 F was obtained on specimens containing 1 percent $CaCl_2$ and cured less than 14 days. The cement used in these tests was a blend of Type II cements bought on the open market. The trends shown in Fig. 21 have been observed on practically all similar tests conducted by the Bureau of Reclamation.

Results of freezing and thawing tests indicate that $CaCl_2$ should be used for early protection of concrete placed at winter temperatures[6,14] but that any

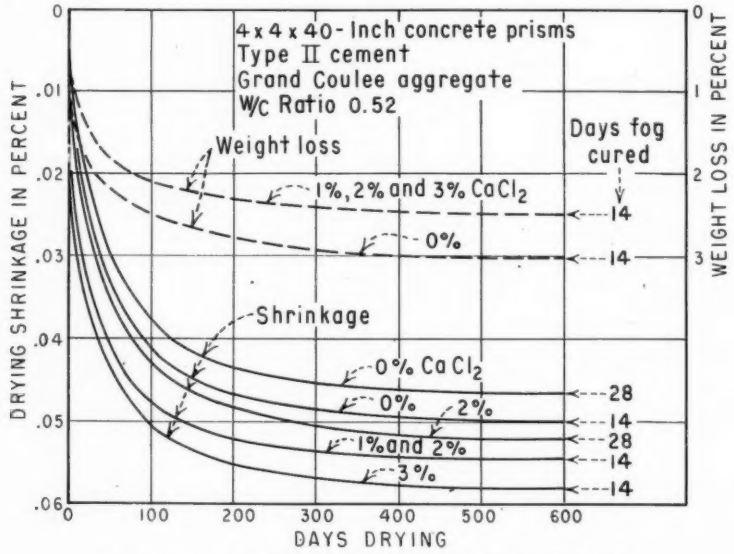

Fig. 22—$CaCl_2$ increases drying shrinkage and reduces weight loss

addition of $CaCl_2$ reduces the durability of older, more completely hydrated, concretes.

Volume change

$CaCl_2$ increases the volume change of concrete under either moist curing or drying conditions. In tests at Grand Coulee[3] with three types of cement and three cement contents, concrete without $CaCl_2$ exhibited about two-thirds the expansion of concrete with $CaCl_2$ in 5 years of storage in water. However, none of the expansions exceeded 0.026 percent and are not considered detrimental.

Drying shrinkage is generally increased by additions of $CaCl_2$ as shown in Fig. 22.[15] Two percent $CaCl_2$ increased the shrinkage about 10 percent for concrete dried for 550 days at 70 F and 50 percent relative humidity. Although drying shrinkage is increased, moisture loss is reduced as shown in this figure. The moisture loss was reduced one-sixth by the addition of $CaCl_2$.

Fig. 23 shows similar data but with considerably greater differences for concrete containing Type V cement. Numerous investigators have found

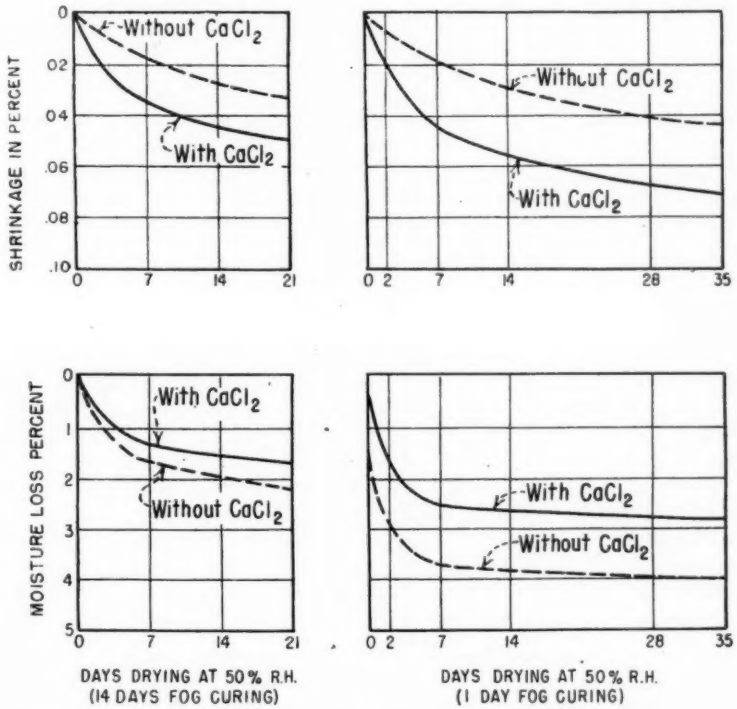

Fig. 23—$CaCl_2$ increases drying shrinkage although the moisture loss is decreased

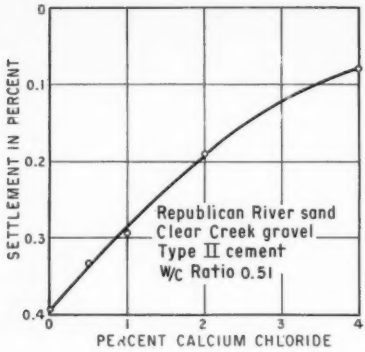

Fig. 24—$CaCl_2$ reduces initial settlement

similar relationships, although Yates[16] found that $CaCl_2$ decreased volume change of concrete both in moist-curing and drying conditions.

Initial settlement

$CaCl_2$ reduces the initial settlement of concrete. A typical curve is shown in Fig. 24. Two percent $CaCl_2$ reduced the initial settlement about one-half, by accelerating the set of the cement. This decrease in the initial settlement allows more voids to remain in the concrete and may account, at least in part, for the greater shrinkage obtained upon drying, as these voids cannot restrain the normal shrinkage of the mortar. The initial settlement of concrete without $CaCl_2$ is about six times the total drying shrinkage possible after the concrete has hardened. Where this settlement is important, as in backfilling horizontal holes, or under horizontal surfaces, the use of $CaCl_2$ in the concrete can be very beneficial.

Time of set

Calcium chloride in amounts normally used accelerates the time of set of cement. Several investigators, particularly prior to 30 years ago, reported that less than 1 percent $CaCl_2$ retarded the set of portland cement. Our tests involving three types of cements and $CaCl_2$ showed that small amounts of the salt did not retard the set. $CaCl_2$ in amounts of 0.1, 0.3, 0.5 and 1 percent accelerated the set by 25, 15, 45 and 85 minutes, respectively. The National Bureau of Standards[17] data show that 0.5, 1 and 2 percent $CaCl_2$ reduced both the initial and final set about ⅓, ½ and ⅔, respectively, for eight, Type I cements. Abrams[4] showed that the set was accelerated as $CaCl_2$ was increased and that more than 4 percent of the salt often produced a flash set.

Piepmeier and Clemmer[18] performed a series of time-of-set determinations at temperatures of 38 and 68 F using 2 and 3 percent $CaCl_2$. Three percent of the salt reduced the time of set from about 10 hours to 1 hour at 38 F and from 3½ hours to 45 minutes at 68 F.

Reinforcing steel in concrete

Many investigators have found that reinforcing steel embedded in dense concrete containing $CaCl_2$ is not damaged by the salt. Corrosion has been found on structural steel embedded in concrete which has been exposed to varying humidity conditions when the concrete is porous or moderately dense. However, in all instances where this corrosion was observed the extent of the action was not progressive and seemed to take place at early ages only. Cottringer and Kendall[9] found no corrosive action on steel bars embedded in concrete containing 0 to 10 percent $CaCl_2$ at the end of a period of 1 year.

J. C. Pearson[20] found no corrosion on steel embedded in concrete containing 2 percent $CaCl_2$ after 6 years' exposure, and Vollmer[21] found that when intimate contact exists between steel and concrete, no corrosive effect is seen to take place. However, steel embedded in high-slump concrete (6 to 8 in.) showed slight rust coloration at early ages but, as borne out by other investigators, this corrosion was not seen to be progressive at later ages. Studies by the Pennsylvania Department of Highways[22] have shown that steel bars immersed in a solution of 2 percent $CaCl_2$ showed less corrosion than similar bars exposed to tap or distilled water.

Erosion resistance

Data obtained in abrasion and cavitation tests conducted in the Bureau laboratories strongly indicate that concretes containing 2 percent $CaCl_2$ are

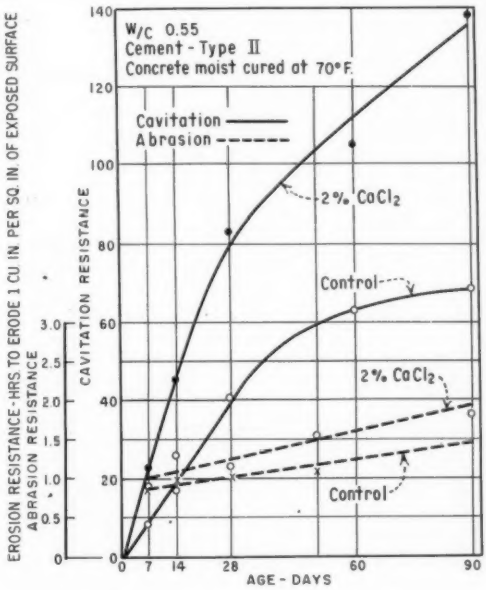

Fig. 25—$CaCl_2$ improves the erosion resistance of concrete

more resistive to erosive forces than plain concrete under either moist curing or drying conditions. Fig. 25 shows that as the curing period was increased, the concretes containing the salt maintained the proportional increase in resistance to erosion. At all ages to 90 days the cavitation resistance of the admixed concrete was about 100 percent greater than the plain concrete, the abrasive resistance was from 10 to 25 percent greater. Vollmer[10] has shown that calcium chloride was beneficial in promoting resistance to abrasion.

CONCLUSIONS

Bureau of Reclamation tests show that $CaCl_2$ in amounts up to 2 percent by weight of cement is often beneficial in concrete. Larger amounts generally produce little additional advantage and are often detrimental. More than 4 percent $CaCl_2$ often produces flash set and lower strengths.

One or two percent $CaCl_2$ accelerates early strength development and this increased strength may persist for several years. $CaCl_2$ may be especially beneficial for concrete exposed to low or freezing temperatures at early ages. It increases the early heat development, accelerates the set, and lowers the freezing point of the concrete slightly. It increases expansion under moist curing; it increases shrinkage but decreases moisture loss under drying conditions. It increases the expansion caused by the alkali-aggregate reaction, and lowers the resistance of concrete to sulfate attack. Freezing and thawing durability is increased at early ages by $CaCl_2$, but is reduced at later ages. It is not detrimental to embedded steel. It increases the resistance of concrete to erosive and abrasive action.

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Discussion of a paper by J. J. Shideler:

Calcium Chloride in Concrete*

By H. C. VOLLMER, G. M. WILLIAMS and AUTHOR

By H. C. VOLLMER†

Mr. Shideler's paper is of particular value to both researchers and construction engineers interested in the use of calcium chloride in concrete because of its wide scope and its correlation with other research work on this subject. Some of the effects of calcium chloride in concrete discussed in the paper have been rather thoroughly established and reported in previous research papers, while other effects discussed are new or relate to subjects on which the available data are limited.

It is of interest to note that the technical literature on the use of calcium chloride in concrete is rather extensive. Since 1870 most of the important concrete investigators have at one time or another been interested in this subject and studies have covered a wide range of properties of cement and concrete under various conditions. This material has recently been collected and reviewed in an annotated bibliography‡ classified by date, author and subject.

A review of early literature indicates that the use of integral calcium chloride in concrete was as an antifreeze or depressant of the freezing point of the mixing water and accordingly the use of relatively high amounts were frequently studied. As mentioned by Mr. Shideler this is not the true function of the additive. Rapp, working at the National Bureau of Standards, showed that calcium chloride accelerates hydration of tricalcium and dicalcium silicates and somewhat retarded hydration of tricalcium aluminates. Foster and Wells showed that the normal chemical process of hydration is not changed. Calcium chloride probably acts somewhat catalytically to accelerate hydration of silicates, the principal strength forming compounds, and combines with aluminates similar to the action of gypsum in cement. Rapp, Abrams and others have shown there is a definite optimum of calcium chloride of about 2 to 3 percent at normal temperatures and that generally 1 to 2 percent may produce satisfactory results.

For practical usage the percentage of calcium chloride is not critical and may be conveniently controlled within the permissible tolerance for other

*ACI JOURNAL, Mar. 1952, *Proc.* V. 48, p. 537. Disc. 48-36 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 24, No. 4, Dec. 1952, Part 2, *Proceedings* V. 48.

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‡Bibliography No. 13 of Highway Research Board.

concrete materials by incorporating it in solution form or using it in the dry form. Except for very dry concrete mixes the use of the dry material is the more practical.

Several valuable effects of the use of calcium chloride in concrete not mentioned by Mr. Shideler are reported in the literature notably by Rapp, Vollmer and Kalousek. These reports show that calcium chloride increases workability of non-air-entraining concrete; also Powers and Steinour have indicated that calcium chloride decreases the bleeding tendency of concretes.

The effect of calcium chloride on the resistance of concrete to freezing and thawing is of importance to many users. The literature reviewed in the bibliography in practically every instance indicates that calcium chloride in air-entraining concrete maintains the high order of durability characteristic of air-entraining concrete. This applies both in studies of change in dynamic E in laboratory freezing cycles and studies of resistance to scaling action of concrete subjected to concentrated applications of chemicals for ice removal such as is the practice in highway winter maintenance work. Field studies of some 100 miles of older concrete roads in New England in which calcium chloride was used in non-air-entraining concrete have definitely shown that these roads have been highly durable and satisfactory after as much as 20 years of service with constant winter ice-removing practice.

Reports presented in the literature indicate that volume change studies of concrete with and without calcium chloride have in many instances produced conflicting indications. Often observations have been based on age readings, that is, a definite length of time after fabrication of specimens, rather than on equal hydration basis which is necessary in studying volume changes when an accelerator of hydration is used.

The present day need for low temperature construction, use of air-entraining and low heat cement, and concrete containing cement blended with large percentages of pozzolanic materials are a few of the factors which have directed interest to calcium chloride as an addition to the concrete mainly for the purpose of insuring curing protection and satisfactory development of early strength. The effect of calcium chloride on the other important properties of concrete under these conditions should be thoroughly understood. It is believed that Mr. Shideler's paper is an excellent source of up-to-date information on this subject. Comprehensive and practical studies such as this paper furnish the materials and construction engineers with basic information.

Referring to Mr. Shideler's comment as to the effect of calcium chloride in concrete to resist sulfate attack, Dalton G. Miller stated in a paper presented at the 1924 meeting of ASTM, "Adding 4 percent calcium chloride to concrete increased the life of specimens in sulfate solutions about 20 percent."

As mentioned by Mr. Shideler, studies conducted by Yates at the Bureau of Standards indicated that calcium chloride reduced volume changes in cement specimens. This study involved a unique test method permitting observations from the time the specimens were fabricated. In any event, there appears to be no practical significance in the numerical differences in volume

changes of cement and concrete specimens with and without calcium chloride. Many years of observations of practical usage have confirmed this opinion. An experimental test road in West Virginia constructed during World War II using air-entraining cement with and without 2 percent calcium chloride showed one-half as many transverse cracks presumably due to drying shrinkage on the calcium chloride section.

The effect of calcium chloride on alkali-aggregate reaction is a relatively recent development of the published literature. However, in 1938 Stanton of California made limited tests which indicated that calcium chloride did not affect the reaction. The alkali-aggregate reaction appears in some measure to be a function of the degree of hydration because it is more often characteristic of older and more completely hydrated cements. In most cases it may take many years to exhibit results. It would be of interest to determine if calcium chloride merely accelerates the phenomena where it may eventually occur rather than numerically increasing its extent. It is believed that the use of low alkali cement or finely ground pozzolanic materials has effectively inhibited the alkali-aggregate reaction with or without the use of calcium chloride.

By G. M. WILLIAMS*

One of the most valuable characteristics of concrete containing $CaCl_2$ is its ability to attain compressive strengths assumed in structural design when cured under adverse conditions of low humidity and high temperatures encountered in field construction. Much work is done in the laboratory to select proportions of cement, aggregate and water under standardized curing conditions but construction practice too often ignores the latter.

Ideal curing conditions are generally impossible in reinforced structural work, with the result that highly stressed members are subjected to early loss of moisture which can never be replaced and the compressive strengths attained are some unknown fractional percentage of the strength assumed in design.

It has been the writer's experience that concretes containing $CaCl_2$, when cured for 24 hr and later exposed to low humidity and high temperature, will attain the strength of similar concrete cured continuously under standardized laboratory conditions. Use of $CaCl_2$ in all structural concrete, where its presence might not be seriously detrimental, would tend to close the gap between design quality of the laboratory and the uncertainty of attainment of specified strength in the structure where curing conditions cannot be controlled.

AUTHOR'S CLOSURE

Mr. Vollmer has made notable contributions on the use and effects of calcium chloride in concrete and his discussion of this paper is appreciated. The additions made both by Mr. Williams and Mr. Vollmer are welcome, as

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the purpose of the original paper was to provide an up-to-date summary of the effects that generally can be expected when $CaCl_2$ is used in concrete.

Our tests have also shown that $CaCl_2$ has increased workability in some non-air-entraining concretes, and that it has been especially effective in increasing workability when used in combination with an air-entraining agent. When $CaCl_2$ is used, the amount of agent is usually reduced to maintain the same air content.

When dry $CaCl_2$ is added to the mix it should be free of lumps and thoroughly mixed into the concrete. One instance of discolored and disrupted concrete in this area was attributed to lumps of $CaCl_2$ remaining near the surface of the finished concrete slab. Bureau of Reclamation specifications require that $CaCl_2$ be added as a solution.

Additional tests are now in progress in the laboratory to supplement previous data on freezing and thawing and sulfate resistance of concrete containing $CaCl_2$. The present trends substantiate the conclusions of the original paper, that $CaCl_2$ in amounts up to 2 percent reduces the resistance of concrete to sulfate attack, and that it is beneficial at early ages but detrimental at later ages to freezing and thawing resistance. There is some indication that when fly ash is included in the mix, $CaCl_2$ improves resistance of moist-cured concrete to freezing and thawing at all ages.

An extensive series of tests to further evaluate the effect of $CaCl_2$ on alkali-aggregate reaction is also in progress. As Mr. Vollmer mentioned, if the deleterious expansion of mortars containing medium or low alkali cement are adequately controlled by additions of pozzolanic materials, the use of $CaCl_2$ is not detrimental. However, even though expansion of mortars containing high alkali cement can be reduced below specification limits by pozzolanic additions, these expansions can be expected to be doubled when 2-percent $CaCl_2$ is used.

A recent report by Dalton G. Miller and Phillip W. Munson* shows that $CaCl_2$ decreased the resistance to sulfate attack of concrete containing a good cement at both normal and elevated temperatures. With less resistant cements and at an elevated curing temperature of 125 F, addition of $CaCl_2$ increased the sulfate resistance of the concretes.

The statement by Mr. Williams that $CaCl_2$ is particularly beneficial in concrete under adverse and inadequate curing conditions may partially explain the apparently superior durability noted by Mr. Vollmer.

*"Long-Time Tests of Concretes and Mortars Exposed to Sulfate Waters," *Technical Bulletin* No. 194, University of Minnesota Agricultural Experiment Station, May 1951.

Title No. 48-37

Reinforcing Steel in Concrete and the Concept of Safety*

By K. HAJNAL-KÓNYI†

SYNOPSIS

Comparative tests on 36 beams reinforced with various types of large size bars, both in ordinary and high grade concrete, prove the superiority of cold worked steel over steel having a natural yield point as regards safety factor and warning before failure. They also prove the advantage of deformed bars over plain bars regarding bond, crack formation and the necessity of increasing the strength of reinforcement, with improved bond, to avoid failure without warning.

Two beams reinforced with 0.104-in. plain wires with a 268,800 psi tensile strength were also tested. Failure occurred by fracture of the reinforcement although the wires were not prestressed. Cracks were much narrower than with large size bars at comparable stresses.

Strain measurements, within a very wide range of steel stresses at failure (42,500 to 294,200 psi), demonstrate good agreement with Whitney's method of determining the position of the neutral axis.

INTRODUCTION

The efficiency of 17 different bars, some having a natural yield point, some cold worked, varying in diameter between 0.71 in. and 1.25 in., was compared in beams 18 in. deep, 8 in. wide, 12 ft long, tested on a span of 11 ft with third-point loading. Each type of bar was tested in concrete with a specified cube (6 in. cube) strength of 3000 psi, marked *L*, and 6000 psi, marked *H*.

The principles of comparing the various types of bars used in the tests (Fig. 1 and 2), the design of the test beams (Fig. 3), the test program, the method of testing, the analysis of the test results and the development of cracks, were described and discussed in *The Structural Engineer*.[1] The tests were performed in two series, 22 beams in one and 14 in the other.

The purpose of the present paper is to (a) report on a third series comprising 2 beams of rather unusual design, (b) compare various types of reinforcement from the standpoint of cracking, (c) describe the results of strain measurements, (d) show the suitability of Whitney's method for assessing the ultimate bending moment of reinforced concrete, (e) investigate the degree of safety obtained with various types of bars on the basis of "Building Code Require-

*Presented at the ACI 48th annual convention, Cincinnati, Ohio, Feb. 27, 1952. Title No. 48-37 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 7, Mar. 1952, *Proceedings* V. 48. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than July 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

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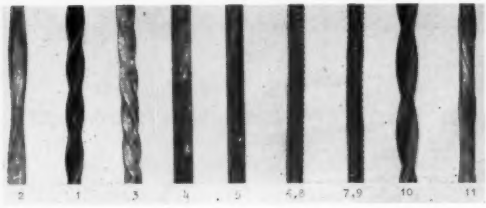

Fig. 1—Bars in Series 1—1—1-in. diameter oval bar twisted to 6 in. pitch. 2—1-in. diameter 12 in. pitch. 3—⅞-in. square twisted bar. 4—1¼-in. diameter mild steel bar. 5—1⅛-in. diameter high tensile bar. 6–8—1-in. diameter American deformed bar. 7–9—1-in. diameter American deformed bar, twisted. 10—¾-in. diameter twin twisted bar. 11—1-in. diameter torsteel bar

ments for Reinforced Concrete (ACI 318-51)," and (f) consider generally the influence of the type of reinforcement on safety.

Notations used in this paper which are not included in the ACI Code are:

f_u = ultimate tensile strength of steel
f_y = yield point of steel
t = steel stress at failure
c = maximum concrete stress at failure
x = depth of neutral axis
a = lever arm

Yield point for cold worked steel represents the stress corresponding to a strain of 0.005, in accordance with British Standard 1144. It would be more correct to call it 0.5 percent proof stress since it has a different meaning than yield point for a hot rolled bar.

Beam marks 1–17 refer to the type of steel used, shown in Fig. 1 and 2. Beams 18 and 19 contained 2 and 3 Tentor bars of 0.71 in. diameter, respectively. Beams 17 and 18 were made only in L concrete.

Test results are analyzed by (a) Whitney's method (symbols indexed p), and (b) the standard method (symbols indexed n). The concrete stress, c_p, in the rectangular stress block has been assumed in the first and third series at 2/3 of the average cube strength. In the second series (beams 12 to 19 inclusive) the factors arrived at from the prism tests were 0.76 for L concrete and 0.64 for H concrete. In England compressive strength tests of concrete are made on 6-in. cubes and cylinder tests are unusual.

In reference 1 the modular ratio is 15, independent of the quality of the concrete, in accordance with the British Code of Practice CP114, 1948. In the present paper L concrete is considered to have a cylinder strength of 2500 psi, H concrete a cylinder strength of 5000 psi. Consequently, the modular ratio is 12 for L concrete and 6 for H concrete. The differences in the steel stresses compared with stresses obtained with $n = 15$ are insignificant, the maximum being 5 percent; but in H concrete, the concrete stresses resulting from the two methods of calculation often differ considerably.

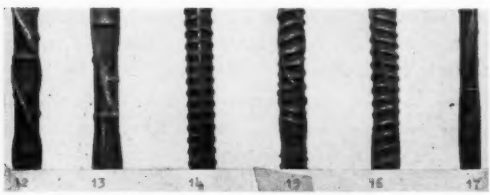

Fig. 2—Bars in Series 2—12—1-in. diameter indented bar twisted to 6 in. pitch. 13—1-in. diameter indented bar twisted to 12 in. pitch. 14—1-in. diameter American deformed bar untwisted (British steel). 15—1-in. diameter American deformed bar twisted to 6 in. pitch. 16—1-in. diameter American deformed bar twisted to 12 in. pitch. 17—0.71-in. diameter Tentor bar

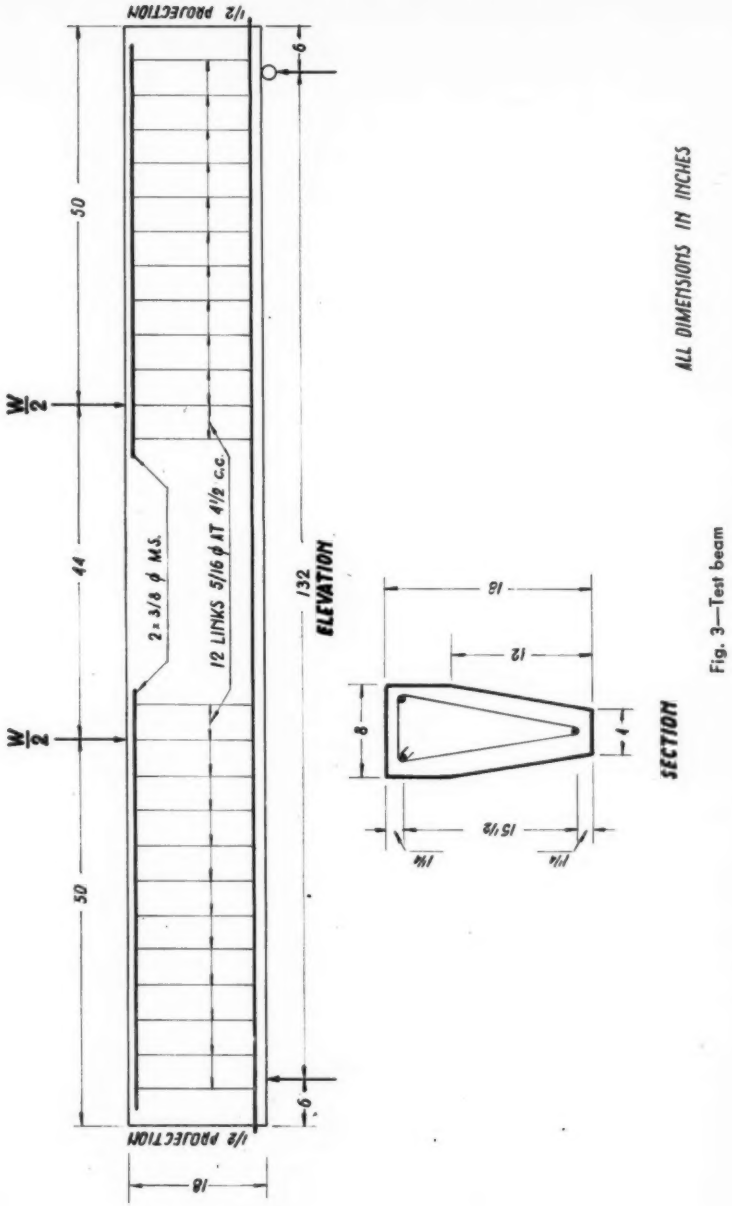

Fig. 3—Test beam

BEAMS REINFORCED WITH HIGH STRENGTH WIRES

Two beams of the cross section shown in Fig. 4, marked 20*L* and 20*H*, were tested to investigate the behavior of untensioned wires of 0.104-in. diameter with a tensile strength of 268,800 psi used as reinforcement for concrete. Such wires are used only in prestressed concrete. The arrangement of the beams was similar to the preceding series shown in Fig. 3. The stirrups shown in Fig. 4 were spaced at 3 in. centers: 17 stirrups were provided at each end over a length of approximately 4 ft 3 in. to avoid failure by shear. No stirrups and no top bars were placed in the central portion 3 ft 6 in. long. The chief aim of the tests was (a) to trace the development of crack widths in un-prestressed concrete at very high steel stresses, (b) to ascertain whether it is possible to reach a steel stress of 268,800 psi at failure if none of the wires is prestressed and the total breaking force of the reinforcement is approximately equal to the yield strength of mild steel at the *economic* percentage in 2500 psi concrete. With $f_s = 18{,}000$ psi, $f_c = 1125$ psi, $p = 0.0134$ and assuming the same effective depth as in beam 20*H*, $A_s = 0.0134 \times 8 \times 16.5 = 1.77$ sq in. With a yield point of 36,000 psi, the yield strength is 63,700 lb. The breaking strength of the wires in the test beams was $28 \times 0.0085 \times 268{,}800 = 64{,}000$ lb. The mix for each of the two beams was different; beam 20*L*, a nominal 1:2:4 mix, by volume, with a water-cement ratio of 0.55, by weight, and a slump of 2½ in., and beam 20*H*, a nominal 1:1½:3 mix with a water-cement ratio of 0.44 and a slump of 2½ in.

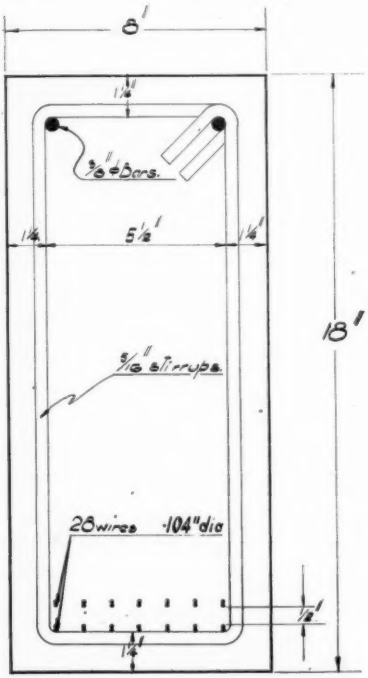

Fig. 4—Cross section of beam 20 (series 3)

Beam 20*L* was to be tested when the concrete cube strength reached 3000 psi. Owing to unforeseen circumstances the test was postponed until the concrete was 41 days old. The cube strength exceeded the specification by 33 percent. In beam 20*H* the better mix was used, that the influence of increased concrete strength might be observed. The average cube strengths of the two

TABLE 1—MAXIMUM CRACK WIDTHS

Load, lb	Width of cracks, mm	
	Beam 20*L*	Beam 20*H*
10,000	0.10 (0.0039 in.)	0
12,000	0.30 (0.0118 in.)	0.20 (0.0078 in.)
14.000	0.40 (0.0157 in.)	0.35 (0.0137 in.)

beams are included in Table 2 along with a summary and analysis of the test results.

The beams and cubes were cured under damp sacks for 24 hours, then in the open air until tested. The beams were supported on rollers, with a span of 132 in., and the load was applied at third points. At the supports and points of loading distribution plates 6 in. wide were inserted between the rollers and the concrete. The load was applied by a jack with a proving ring to record 2000-lb increments. The deflection at midspan was measured by a dial micrometer, and similar instruments were set at each support to correct for settlement.

Measurements of the change of strain of the concrete were taken at nine positions on each side of the beam at the center section. A special extensometer was used with a gage length of 10 in. (Fig. 5). The gage points consisted of pairs of small steel plates, each approximately ½ in. square, attached to the concrete by quick-setting glue. A 0.04 in. diameter hole was drilled in each plate and the points of the micrometer fitted into them. The accuracy of reading was 0.0001 in. so that strains of 1×10^{-5} could be measured.

The beams were examined for cracks at every loading stage and the strain measured at every second loading stage, *i.e.*, at increments of 4000 lb.

After applying a total of 20,000 lb the load was released and the permanent deflection, crack widths and residual strain of the concrete observed.

Fig. 6 shows the load-deflection diagram in which the maximum crack widths at various stages of loading are indicated. In this diagram beams 4*H*, reinforced with one 1¼-in. diameter mild steel bar, 6*L* reinforced with one American deformed 1-in. diameter mild steel bar, 14*L* reinforcement similar to that of 6*L* but bar rolled in England, and 18*L* reinforced with two 0.71-in. diameter Tentor bars, from the preceding series are also included.

Fig. 6 shows that no cracks occurred in beam 20*L* at a load of 8000 lb or in beam 20*H* at a load of 10,000 lb. At 10,000 lb on beam 20*L* and 12,000 lb on beam 20*H*, 4 and 5 cracks, respectively, were observed. The maximum crack widths at the early stages of cracking are listed in Table 1.

Despite the very low percentage of reinforcement, the maximum width of cracks was not excessive. By inter-

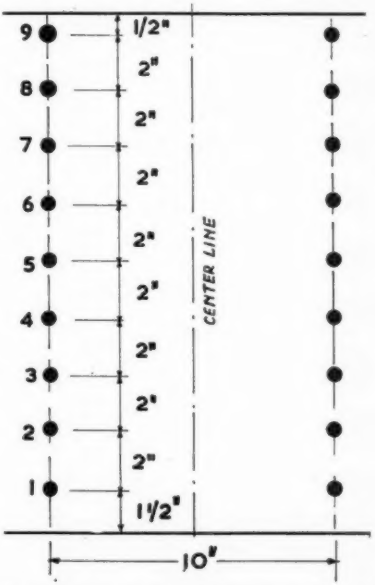

Fig. 5—Position of strain gages on front of beam. Gage positions 11-19 on rear of beam identical with 1-9 shown here

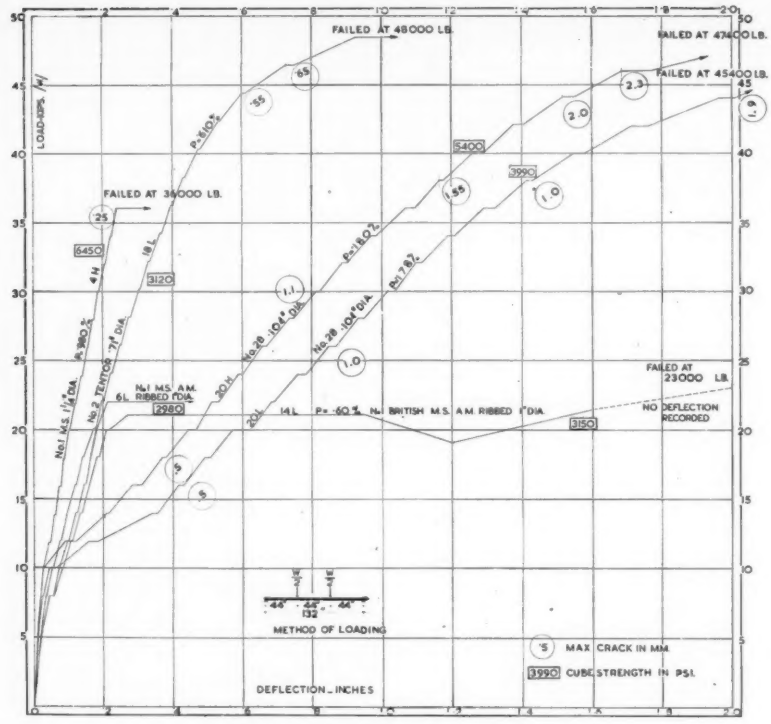

Fig. 6—Typical load-deflection diagrams

polation, the steel stresses corresponding to a maximum crack width of 0.25 mm (0.0098 in.) are: 74,900 psi in beam 20*L* and 81,600 psi in beam 20*H*.

These are rough approximations since the accuracy in measuring crack widths was ±0.03 mm (0.0012 in.), but they indicate clearly the favorable influence of the small wires on the cracks, in spite of their smooth surface.

If such wires were used in ordinary reinforced concrete, the limiting working stress would not be governed by consideration of safety against failure but by crack width and deflection.

Further enlargement of cracks can be seen from Fig. 6. A maximum width of 0.5 mm (0.0196 in.) was reached in beam 20*L* at 16,000 lb, in beam 20*H* at 18,000 lb. Before failure, the widest cracks were about 2 mm (0.0787 in.). Fig. 7 shows the central part of beam 20*H* at the last loading stage. The number of cracks increased at every loading stage. Finally 37 cracks were counted, with an average spacing of approximately 3½ in. Many of the cracks were very fine, even immediately before failure, and only a few cracks approached 2 mm (0.0787 in.).

After the release of the load of 20,000 lb, the residual deflections were: beam 20*L* 0.248 in., and beam 20*H* 0.099 in. They represent 39.2 and 20.8 percent, respectively, of the total deflection at 20,000 lb, which was only 44 and 42 percent, respectively, of the maximum load. These figures show that recovery was inadequate. Thus, the tests confirm the well-known fact that untensioned wires of very high strength cannot be used alone, economically, as main reinforcement.

TABLE 2—ANALYSIS OF TEST RESULTS—SERIES 3

Beam No.	20*L*	20*H*
Reinforcement	High tensile wires No. 28, 0.104 in. diameter	
Sectional area, A_s, sq in.	0.238	0.238
Ultimate strength, f_u, psi	268,800	268,800
Distance of center of gravity of steel from bottom, in.	1.25	1.5
Effective depth, d, in.	16.75	16.5
Percentage reinforcement, 100 $A_s/8d$	0.178	0.180
Average cube strength, psi	3990	5400
Maximum load, w, lb	45,400	47,400
Maximum bending moment, M, lb-in. $\times 10^{-6}$	1026	1070
Maximum M/bd^2c_p	0.172	0.136
Ratio a_p/d	0.905	0.926
Depth of neutral axis, x_p, in.	3.183	2.442
Lever arm, a_p (plastic theory), in.	15.159	15.279
Steel stress at failure, t_p, (plastic theory), psi	284,400	294,200
Lever arm, a_s (standard method), in.	15.71	15.75
Steel stress at failure, t_s (standard method), psi	274,400	285,450
Ratio, t_p/f_u	1.058	1.094
Stress at which maximum crack width = 0.25 mm, psi	74,900	81,600
Number of cracks at failure	37	37

In the discussion of a preceding paper[1] Abeles drew a diagram showing the effect of partial and full prestressing on a beam having the same reinforcement as beams 20. He demonstrated that partial prestressing would render such beams a practical proposition. Failure of both beams occurred through fracture of the reinforcement (Fig. 8).

COMPARISON OF BARS FROM STANDPOINT OF CRACKING

Table 3 compares the interpolated stresses at which the maximum crack width was 0.25 mm (0.0098 in.). Although this width is arbitrary it is a good basis for comparing the relative merits of various types of bars. Table 3 summarizes all 3 series and includes a further series carried out on similar lines with welded fabric made of hard drawn wire.

The bars are divided into four groups. The first contains *large* size bars, *i.e.*, between 1 and 1¼-in. diameter or equivalent cross section. The second

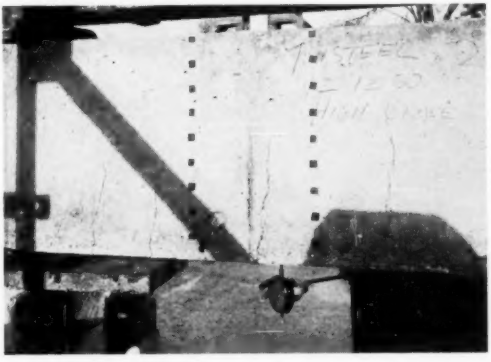

Fig. 7—Beam 20*H* under load of 46,000 lb

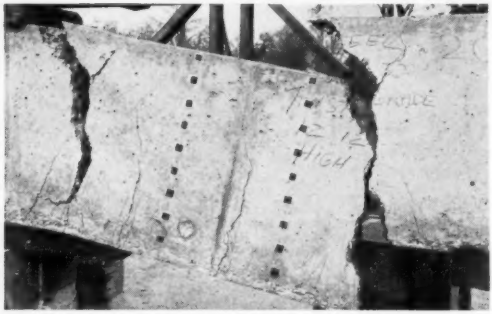

Fig. 8—Beam 20H after failure

group comprises the 0.71-in. diameter Tentor bars, the third hard drawn wires 0.25 to 0.4-in. in diameter, and the fourth high tensile wires of 0.104-in. diameter.

Note that the first five bars in the first group have a plain surface. Among the plain bars, the mild steel bar, owing to its larger diameter, gives the smallest stress; among the deformed bars the American type gives the highest stress.

The difference between 1-in. indented twisted bars and 0.71-in. Tentor bars clearly indicates the influence of the size since the shape of these two bars is very similar.

The most striking feature of Table 3 is the large increase in stress in groups three and four. Wires, included in these groups, are of plain round cross section and have a smoother surface than the heavier hot rolled bars. Table 3 also shows the advantage of using as many small bars as possible instead of a small number of large bars and also shows the favorable effect of corrosion.

TABLE 3—STRESS AT MAXIMUM CRACK WIDTH = 0.25 mm.

Reinforcement	Stress, psi	No. of beams tested
1¼-in. mild steel	27,900–44,800*	2
¾-in. twin twisted	33,100–38,200	2
1⅛-in. high tensile	35,400–42,800	2
⅞-in. square twisted	37,600–52,000†	2
1-in. oval twisted	38,500–49,200	4
1-in. indented twisted	41,600–43,400	4
1-in. Torsteel	41,900–47,600	2
1-in. American ribbed deformed	42,300–49,700	10
0.71-in. Tentor	46,900–62,500	4
0.4-in. hard drawn wire, smooth	51,600–55,900	3
0.4-in. hard drawn wire, corroded	57,800–73,500	3
0.25-in. to 0.324-in. hard drawn wire, smooth	58,600–67,200	6
0.104-in. wire	74,900–81,600	2

*Concrete of 6400 psi cube strength
†Concrete of 6200 psi cube strength

STRAIN MEASUREMENTS

Strain readings, described previously, were taken on 9 beams of Series 2 and on both beams of Series 3. Fig. 9, 10 and 11 show a few characteristic results. The plotted values are the averages obtained from readings on both faces of the beams. The numbers on the top and bottom of the figures indicate the load in kips. Where the strain is negative (tension) most measurements were taken over cracks. Negative strains exceeding the maximum at which the concrete cracks represent the total elongation

of the concrete, which is very small, plus the width of cracks between the gage points divided by the gage length. Therefore they are not actual *strains*. It depends on the position and shape of the cracks within the gage length whether or not the straight line relationship is maintained, as in Fig. 11. Sometimes a crack was within the gage length in the lower part of the beam and outside the gage length higher up (Fig. 10).

The change of position of the neutral axis between two consecutive loading stages is very small. It seems justified to consider the depth obtained at the last loading stage as the depth at failure.

For comparison, the depths obtained by the standard method, x_n, and by the assumption of a rectangular stress block, x_p, are indicated in the diagrams.

Fig. 9 shows the effect of yielding of mild steel in beam 14*L*. An upper and lower yield point were distinguishable during the test. Yielding caused the neutral axis to rise considerably. There are two strain lines corresponding to a load of 21,000 lb, one showing the strain immediately after reaching this load, 21 min., the other, 21 max., showing the strain just before the load began to drop to 19,000 lb. It was then possible to increase the load to 23,000

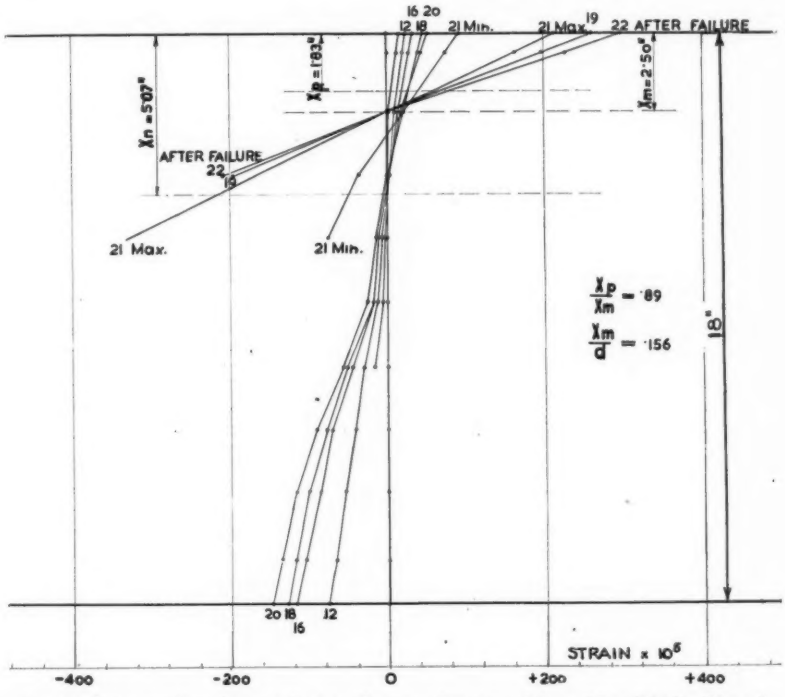

Fig. 9—Strain readings, beam 14*L*, 1-in. diameter American deformed bar, British mild steel

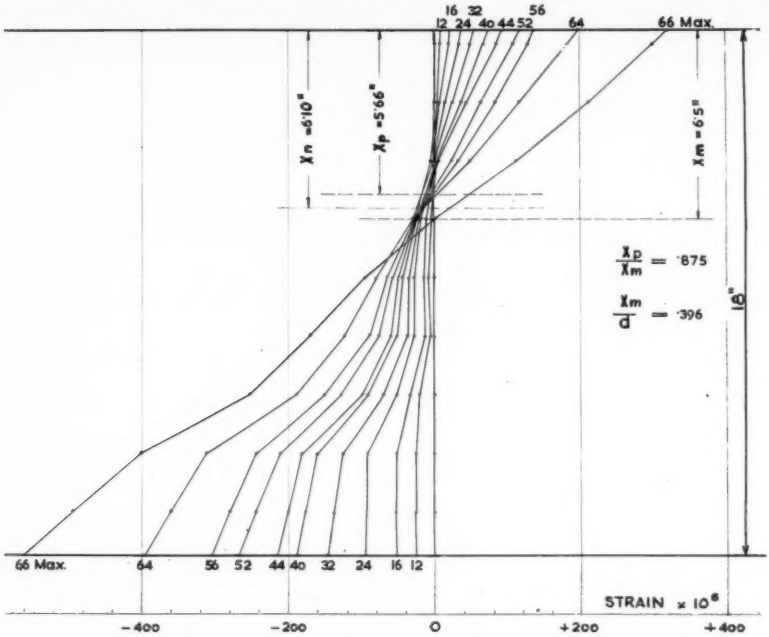

Fig. 10—Strain readings, beam 19*L*, No. 3, 0.71 in. diameter Tentor bars

lb at which point the concrete on the top of the beam was crushed. The beam was still capable of supporting a load of 22,000 lb (see Fig. 6 and its discussion on p. 578).

In beam 19*L*, the nearest to balanced design in all three series ($M/bd^2c_p = 0.285$), there was a downward movement of the neutral axis with increasing load (Fig. 10). In beam 20*H*, reinforced with nonyielding high tensile wire, there was hardly any movement at all, apart from the initial stages (Fig. 11). The strain distribution conforms to a straight line pattern.

It should be noted that the maximum strain observed is about 0.003 units in both beams 14*L* and 19*L* whereas in beam 20*H* (high grade concrete) it was only about 0.0023 units. This difference between *L* and *H* concrete was rather consistent. Greater ultimate strains might have been obtained with over-reinforced beams.

APPLICATION OF WHITNEY'S THEORY

Fig. 12 indicates the relation between the position of the neutral axis and the lever arm according to; (a) the standard method, (b) the probable actual distribution of stress at failure, and (c) Whitney's simplified assumption.

The exact shape of the stress distribution in (b) is not known, but once the position of the neutral axis, x_m, has been determined by measurement

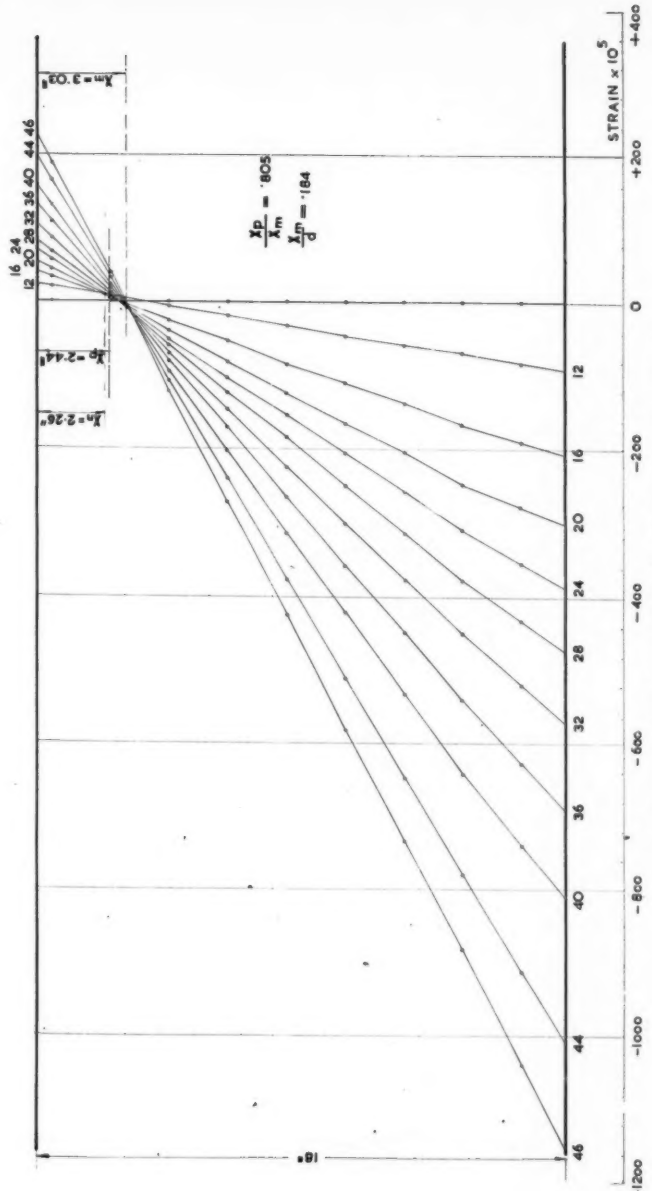

Fig. 11—Strain readings, beam 20*H*, No. 28, 0.104 in. diameter wires

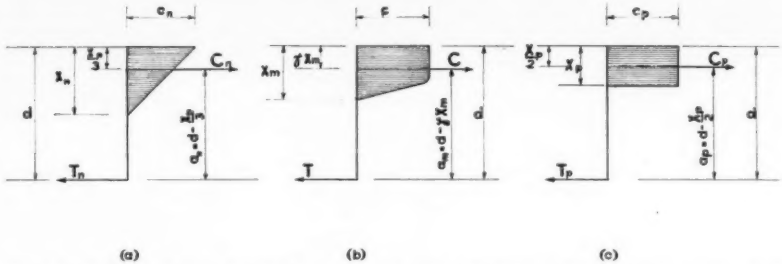

Fig. 12—Position of neutral axis with various stress distributions in the concrete. a—Standard method. b—Probable distribution before failure. c—Whitney's simplified assumption

as in the present tests, the only unknown in the equation $a = d - \gamma x$ is the coefficient γ. It is easy to see that $0.333 < \gamma < 0.500$ and that the actual range of variation is smaller.

Table 4 summarizes the results for 11 beams on which the strain was measured and compares the depth of the neutral axis obtained by calculation, x_p, x_n, with the depth obtained from strain measurements, x_m.

According to Fig. 12, $x_p < x_m < x_n$. The first condition is fulfilled in Table 4. The ratio x_p/x_m, line 9, varies between 0.732 and 0.935. Considering that c_p on which x_p depends was estimated from cube, prism or cylinder tests and the accuracy of the measured value of the depth of the neutral axis is limited, the agreement between measured and calculated values is very satisfactory.

Conditions are quite different for x_n. The ratio x_n/x_m varies between 0.746 and 2.291. This may be explained as follows: the depth of the neutral axis at failure depends not only on the percentage, p, and the quality of the concrete (represented by the modular ratio) but also on the steel stress at failure, t, since it is the *tensile force* in the steel which has to be balanced by the compressive force in the concrete. The standard method ignores the influence of the *steel stress*. As long as the quality of the steel does not vary much, it is not so conspicuous, but with t_p varying between 44,100 and 294,200 psi as in Table 4, this feature of the standard method reveals itself.

In the case of mild steel, beams 14L and 14H, with t_p below 45,000 psi, $x_n/x_m > 2$. With high tensile steel, t_p varies between 68,800 and 85,900 psi, beams 12L, 12H, 13L and 15H, and x_n/x_m varies between 1.269 and 1.576. With Tentor bars where t_p is between 92,300 and 95,400 psi, x_n/x_m varies between 0.938 and 1.002 and with high tensile wires, beams 20L, 20H, where t_p = 284,400 and 294,200 psi, the corresponding values of x_n/x_m are 0.780 and 0.746.

The calculated concrete stresses, c_n, vary between 1820 and 5230 psi in L concrete and between 2370 and 7520 in H concrete. In 14H the concrete was actually crushed at c_n = 2370 psi, *i.e.*, only 42 percent of the cube strength; when the steel yielded. In 20H at c_n = 7520 psi, *i.e.*, 1.39 times cube strength, it was the steel which fractured. Fig. 8 does not show any sign of concrete

TABLE 4—COMPARISON OF CALCULATED AND MEASURED POSITION OF NEUTRAL AXIS

Beam No.	12*L*	12*H*	13*L*	14*L*	14*H*	15*H*
Reinforcement	1-in. Indented twisted			1-in. American deformed		
				Mild steel	Mild steel	Twisted
p, percent	0.565	0.570	0.569	0.607	0.609	0.580
M/bd^2c_p	0.170	0.125	0.153	0.107	0.073	0.121
$p\ t_p/c_p = k_p = x_p/d$	0.188	0.134	0.166	0.114	0.076	0.130
d, in.	16.150	16.000	16.150	16.050	16.000	16.600
x_m, measured, in.	3.250	2.900	3.300	2.500	1.650	2.430
x_p, calculated, in.	3.040	2.140	2.680	1.830	1.220	2.160
x_n, calculated, in.	4.960	3.680	4.970	5.070	3.780	3.830
$k_m = x_m/d$	0.201	0.181	0.204	0.156	0.103	0.146
x_p/x_m	0.935	0.738	0.812	0.732	0.739	0.889
x_n/x_m	1.526	1.269	1.506	2.028	2.291	1.576
t_p (plastic theory), psi	81,700	85,900	68,800	44,800	44,100	83,800
t_n (standard method), psi	82,400	86,800	70,300	47,200	46,000	84,900
c_n (standard method), psi	3,030	4,300	2,600	1,820	2,370	4,270
a_p, in.	14.630	14.930	14.810	15.130	15.390	15.520
$(d-a_p)/x_m = x_p/2x_m$	0.467	0.369	0.406	0.361	0.370	0.444

Beam No.	18*L*	19*L*	19*H*	20*L*	20*H*
Reinforcement	0.71-in. Tentor			0.104-in. high tensile	
	No. 2	No. 3	No. 3	No. 28	No. 28
p, percent	0.610	0.914	0.936	0.178	0.180
M/bd^2c_p	0.211	0.285	0.219	0.172	0.136
$p\ t_p/c_p = k_p = x_p/d$	0.240	0.344	0.250	0.190	0.148
d, in.	16.440	16.440	16.040	16.750	16.500
x_m, measured, in.	5.200	6.500	4.550	4.000	3.030
x_p, calculated, in.	3.950	5.660	4.010	3.180	2.440
x_n, calculated, in.	5.200	6.100	4.560	3.120	2.260
$k_m = x_m/d$	0.316	0.396	0.284	0.238	0.184
x_p/x_m	0.760	0.872	0.880	0.795	0.805
x_n/x_m	1.000	0.938	1.002	0.780	0.746
t_p (plastic theory), psi	93,300	92,300	95,400	284,400	294,200
t_n (standard method), psi	91,800	87,200	92,300	274,400	285,500
c_n (standard method), psi	3,540	4,300	6,080	5,230	7,520
a_p, in.	14.470	13.620	14.040	15.160	15.280
$(d-a_p/x_m = x_p/2x_m$	0.380	0.436	0.440	0.397	0.402

crushing. Compared with cylinder strength, the excess was at least 50 percent. Such an excess has often been found in beams with a high percentage of reinforcement, *i.e.*, low steel stress at failure. Hadley[2] put forward the theory that flexural strength is 40 to 75 percent greater than cylinder strength, the excess varying inversely with the concrete strength. In the present case, the reinforcement was only 0.18 percent. In 20*L* the steel also fractured at a calculated stress higher than its ultimate strength, as obtained from tensile tests. It is safe to conclude that the steel in 20*H* would have fractured even with a substantial increase in the percentage, and the excess of c_n over cylinder strength would have been much greater than 50 percent for concrete of $f_c' = 5000$ psi. Thus, Hadley's hypothesis would fail to explain the load-bearing capacity of beams of the 20*H* type with higher steel percentages and would not allow the establishment of a boundary between under-reinforced and over-reinforced beams. Beams reinforced with high strength wire can only be used in practice if some of the wires are prestressed. Conditions at failure are very similar with and without prestressing and the establishment of such a boundary is therefore of great practical importance.

Admittedly, a permissible concrete stress $f_c = 0.45\,f_c'$ is generous and takes into account the excess of the calculated flexural strength over the cylinder

strength. It should be realized, however, that if f_c is kept constant and f_s increased, x_n is reduced and consequently the limiting factor M/bd^2 up to which no compression reinforcement is required, is also reduced. Table 4 shows clearly that this is an artificial limitation of the use of high tensile steel. As long as the variation of f_s is only between 18,000 and 20,000 psi, the difference is not very important but if higher stresses, f_s, were adopted for certain types of higher quality bars included in the author's tests, the present method of design would greatly reduce, without experimental justification,[3] the saving obtainable with higher quality steel.

This discrimination against high tensile steel can be avoided by a design method not based on permissible stresses, *e.g.*, Whitney's method which covers the whole range of steel stresses included in Table 4. This can best be seen from the last line, containing the values $d-a_p/x_m = x_{pm}/2x_m$, *i.e.*, the distance of the resultant compressive force in the concrete from the top calculated by Whitney's method related to the measured depth of the neutral axis. The figures vary between 0.361 and 0.467; well within the theoretical limits of 0.333 and 0.500. In all the cases where strain measurements were taken, Whitney's assumption was found to be basically correct independent of the quality of steel. If a_p agreed with the actual value a, $d-a_p/x_m$ would be equal to γ. The effect of a discrepancy in $d-a_p/x_m$ on the steel stress increases with increasing M/bd^2c_p. The effect of a possible error in a_p due to the uncertainty of c_p is far greater in $19L$ with $M/bd^2c_p = 0.285$ than in any other beam of the three series, since M/bd^2c_p was substantially smaller in all other beams. If we assume for $19L$, $d-a_p/x_m = 0.37$ (most unlikely) as against 0.436 in Table 4, the difference in a_p is only 0.43 in. or 3.15 percent. A similar assumption for $12L$ results in a difference of 0.315 in. or 2.15 percent for a_p. Therefore it is reasonable to expect that the deviation of the actual steel stress at failure from t_p as given in Tables 4 and 5 is probably less than 2 percent.

In the stress analysis of the 38 beams included in the three series, the value of c_p has been assumed at 0.64 to 0.76 times the average cube strength on the basis of prism tests. This is a comparatively small variation since 0.667 has been assumed for *all* beams in Series 1 and 3, 0.64 for *all* beams in *H* concrete and 0.76 for *all* beams in *L* concrete in Series 2. If an average of 0.7 had been assumed throughout, the difference in the $p(t_p/c_p)$ values would not have been more than ± 10 percent and in the steel stress, t_p, not more than ± 2 percent.

Reverting to the standard method, the variation in the quality of steel could only be taken into account by a completely new interpretation of the modular ratio. Instead of being the ratio of the moduli of elasticity, it would have to be a figure depending on the yield point or ultimate strength of the steel, whichever may be the decisive factor at failure, and on the strength of the concrete.

From evidence in previous publications and from results obtained in tests of these 38 beams it is justifiable to conclude that Whitney's method is a safe

and suitable way to assess the ultimate bending moment of beams of any known quality of steel and concrete. Still to be confirmed by further tests is whether the limit of $M/bd^2f_c' = 0.333$ can be reached with steel of high strength such as used in beams 20.

In contrast, the standard method (1) fails to explain the load-bearing capacity of over-reinforced concrete beams without the assumption of a flexural strength 40 to 75 percent greater than the cylinder strength, (2) would have to be radically modified if it were to be applied to steel of very high strength as used in prestressed concrete, and (3) fails to account for the efficiency of steel in compression unless the modular ratio is multiplied by an arbitrary factor (see Clause 706(b), ACI Code 318-51).

FACTORS OF SAFETY ON THE BASIS OF ACI 1951 BUILDING CODE

Table 5 summarizes stresses at failure in Series 1 and 2 as obtained by Whitney's method and safety factors using the standard method and permissible stresses in accordance with ACI 318-51.

Columns 3 and 4 of Table 5 contain the calculated stresses t_p; columns 11 and 12, t_n. In columns 5 and 6 t_p is related to the yield point of the reinforcement and in columns 8 and 9 to the ultimate strength. Columns 7 and 10 indicate the effect of the concrete quality on these ratios.

The beams have been divided into two groups: *a* reinforced with bars having a natural yield point, and *b* reinforced with cold worked bars.

In each group the beams have been listed according to the ratio t_p/f_y in L concrete. The two groups are distinctly separated by the value of f_p. In group *a*, t_p varies between 42,500 and 56,200 psi, in group b, between 63,400 and 95,400 psi.

All values t_p/f_y are greater than 1 with the exception of the oval twisted bar which began to slip in L concrete just before its yield point was reached. In H concrete the bond was sufficient to exceed the yield point considerably.

The ratios H/L for t_p/f_y are all greater than 1 with the exception of beam 6. This is due to an over-estimation of the load on beam $6L$. This beam failed at an applied load of 22,000 lb immediately upon reaching the load, while in beam $6H$ the maximum load was maintained for 11 minutes.

The effect of concrete quality is less pronounced in group *a* where H/L is below 1.025 than in group *b* where H/L varies from 1.007 to 1.165.

The values of t_p/f_u show that with steel having a natural yield point the stress at failure was between 60 and 70 percent of the ultimate strength. The H/L values are not characteristic since failure is governed by f_y and the ratio f_y/f_u varies from bar to bar.

Group *b* gives a different picture. Here we have to exclude $1L$ (bond slip) $2H$, $7H$, $9L$ and $9H$ because of inadequate development by cold work. The ratio t_p/f_u varies between 0.863 and 0.958 in L concrete and between 0.934 and 1.002 in H concrete. H/L varies between 1.019 and 1.161. In H concrete the ultimate strength is closely approached in the majority of beams.

TABLE 5—COMPARISON OF STRESSES AT FAILURE AND FACTORS OF SAFETY TO ACI 1951 BUILDING CODE

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Beam No.	Description of bars	f_p, psi		f_p/f_y			f_p/f_u			f_n, psi		Factor of safety		L 2.426	H 2.426
		L	H	L	H	H/L	L	H	H/L	L	H	L	H		
	a—Bars having natural yield point														
8	American deformed H.T., 1-in. diameter	52,000	52,900	1.001	1.026	1.025	0.627	0.663	1.057	53,300	55,100	2.665	2.755	1.099	1.136
5	H.T., 1⅛-in. diameter	56,200	55,500	1.019*	1.028	1.009*	0.611	0.605	0.990	57,000	57,600	2.848	2.880	1.174	1.187
6	American deformed mild steel, 1-in. diameter	42,900	45,400	1.069**	1.040	0.973**	0.662	0.699	1.056	44,800	47,700	2.490	2.649	1.026	1.092
14	American deformed, British mild steel, 1-in. diameter	44,800	44,100	1.082	1.100	1.017	0.665	0.667	1.003	47,200	46,000	2.624	2.556	1.082	1.054
4	Mild steel, 1¼-in. diameter	42,800	42,500	1.110	1.122	1.011	0.656	0.636	0.970	43,700	44,800	2.426	2.487	1.000	1.025
	b—Cold worked bars														
1	Oval twisted, 6-in. pitch	72,200†	84,800	0.985†	1.148	1.165†	0.852†	0.989	1.161‡	72,300	86,400	3.614	4.318	1.490	1.780
3	Square twisted, ⅞-in.	69,700	69,100	1.013	1.098	1.084	0.876	0.958	1.094	69,200	71,200	3.458	3.562	1.425	1.468
11	Torsteel, 1-in. diameter	70,200	80,200	1.044	1.133	1.085	0.863	0.952	1.103	70,200	81,900	3.512	4.093	1.448	1.687
18	Tentor, 0.71-in. diameter	93,300	—	1.075	—	—	0.917	—	—	91,800	—	4.591	—	1.892	—
19	Tentor, 0.71-in. diameter	92,300	95,400	1.075	1.109	1.032	0.917	0.934	1.019	87,200	92,300	4.362	4.612	1.798	1.901
15	American ribbed, British mild steel, 1-in. diameter, 6-in. pitch	75,100	83,800	1.082	1.192	1.102	0.906	1.002	1.106	75,600	84,900	3.778	4.243	1.557	1.749
17	Tentor, 0.71-in. diameter	92,400	—	1.088	—	—	0.937	—	—	94,000	—	4.697	—	1.936	—
2	Oval twisted, 12-in. pitch	73,000	68,500‡	1.098	1.120‡	1.020‡	0.942	0.907‡	.963‡	72,800	70,700	3.639	3.534	1.500	1.457
10	Twin twisted, ¾-in. diameter	66,400	65,500	1.115	1.151	1.032	0.915	0.946	1.034	67,200	67,500	3.361	3.377	1.385	1.392
12	Indented twisted, 1-in. diameter, 6-in. pitch	81,700	85,900	1.116	1.166	1.045	0.958	0.999	1.043	82,400	86,800	4.120	4.340	1.698	1.789
13	Indented twisted, 1-in. diameter, 12-in. pitch	68,800	69,600	1.121	1.150	1.026	0.942	0.993	1.054	70,300	71,600	3.514	3.579	1.448	1.475
16	American deformed, British mild steel, 1-in. diameter, 12-in. pitch	72,800	77,400	1.148	1.207	1.051	0.918	0.950	1.035	72,700	78,700	3.635	3.933	1.498	1.621
9	American deformed, H.T., 1-in. diameter, twisted	75,200	73,600‡	1.149	1.157‡	1.007	0.827	0.881‡	1.065‡	73,400	75,700	3.670	3.783	1.513	1.559
7	American deformed, mild steel, 1-in. diameter, twisted	63,400	72,600	1.189	1.213	1.020§	0.896	0.923	1.030	63,800	74,500	3.191	3.724	1.315	1.535

*Slipped at a stress equal to 91 percent of the yield point.
†Failed in bond in L concrete.
‡Undertwisted in H concrete.
§Unsatisfactorily developed.
**See discussion in text.

The difference between the two groups is even more striking when the factors of safety are considered. These are the ratios f_n/f_s. f_s is not included in the table, it has been assumed at 18,000 psi for bars 4, 6, 14 and 20,000 psi for all others. The lowest value, 2.426, was obtained in $4L$. To facilitate the comparison, this figure has been taken as unity in columns 15 and 16. The variation in group *a* is within a limited range, the maximum value being 1.187 for $5H$. In group *b*, $7L$ is again excluded since it was not sufficiently developed. The figures vary between 1.385 and 1.936. Excluding the twin-twisted bar 10, the range of variation for single twisted bars is 1.425 to 1.936.

Thus, for single twisted bars the ACI Code requires substantially higher factors of safety than for bars having a natural yield point.

GENERAL CONSIDERATIONS OF SAFETY

The factor of safety in itself, *i.e.*, the ratio $M_{maximum}/M_{permissible}$ is not a sufficient measure of the safety of a structure. It is important to consider also the way in which a structure fails. Three types of failure may be distinguished (Fig. 13); (a) sudden (or brittle) without any warning, (b) yielding, with a sudden increase of the deformation on reaching the maximum load which can be maintained until the deformation is substantial, and (c) gradual (or tough) with ample warning of the approaching danger, allowing an increase of load even at a stage when failure appears imminent.

Sudden failure is characteristic of cast iron, concrete columns reinforced with mild steel (but not with cold worked bars or with spiral reinforcement), and concrete beams which are either over-reinforced or have such a low percentage of steel that failure occurs at first cracking. Neither of these cases was represented in the present tests. Bond slip and shear failure may also be sudden, but well-designed reinforced concrete beams cannot fail in this way.

Yielding failure is the characteristic form in concrete beams reinforced with steel having a natural yield point while gradual failure is characteristic of concrete beams reinforced with cold worked bars.

The difference between yielding and gradual failure is demonstrated in Fig. 6 by a few characteristic examples. Beams $4H$, $6L$ and $14L$ represent yielding failure; beams $18L$, $20L$ and $20H$ gradual failure. In both beams $4H$ and $6L$, yielding of the reinforcement took place without previous warning.

Beam $4H$ was loaded to 36,000 lb. The deflection was 0.243 in. = span/543, the maximum crack width 0.25 mm (0.0098 in.). Both the deflection and the crack widths indicated the beam to be quite *safe*. After 6 minutes of sus-

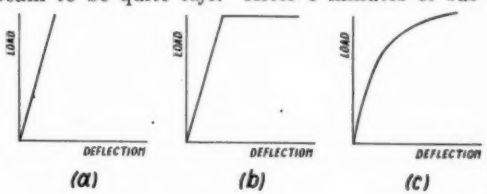

Fig. 13—Classification of structures according to the type of failure

Fig. 14—Beam 4*H* after failure, 1¼-in. diameter plain mild steel bar

tained loading one wide crack appeared, the deflection increased rapidly and the concrete in the top was crushed (Fig. 14).

Beam 6*L* was loaded to 22,000 lb. The deflection was 0.214 in. = span/617, but began to increase immediately owing to yielding of the steel and the top concrete was crushed (Fig. 15). There was no time to take readings of crack widths. At the preceding loading stage, *i.e.*, 20,000 lb, the maximum crack width was only 0.15 mm (0.0059 in.). The difference in Fig. 14 and 15 should be noted. With the plain bar in beam 4*H* one big crack occurred, with the ribbed bar in 6*L* there are two main cracks with several fine branches.

Beam 14*L* was a special case. The reinforcement was similar in shape to 6*L* and the yield point was approximately the same, but the range of yielding in the stress-strain diagram was much shorter. This short range and the excellent bond, made it possible to observe the oscillation of the load between the upper and lower yield point and reach failure at an applied load which was 9.5 percent higher than the load on reaching the upper yield point. Beam 14*H* behaved in the same way. The tensile zone of the central portion of this beam completely disintegrated and fell down when the beam was removed from the machine (Fig. 16). The essential features of this picture are six wide cracks, three on either side of the crack where failure occurred.

Fig. 15—6*L* after failure, 1 in. diameter mild steel American deformed bar

Fig. 16—Beam 14*H* after failure, 1-in. diameter American deformed bar, British mild steel

The extension of the central crack to the top occurred when the beam was taken down from the machine. This formation of cracks after yielding was characteristic of all beams reinforced with American deformed bars.

The disintegration of the tensile zone was a feature of all beams with twisted American deformed bars (Fig. 17), while with twisted Indented bars and Tentor bars only vertical cracks occurred (Fig. 18).

The load-deflection line of beam 18*L* in Fig. 6 is representative of the behavior of beams reinforced with cold worked bars. At a load of 44,000 lb the deflection was 0.6 in. = span/220 and the maximum crack width was 0.55 mm (0.0216 in.). The beam appeared to be in a dangerous condition yet the load increased to 48,000 lb before the concrete was crushed.

Beams 20*L* and 20*H* are examples of *too much warning*. Owing to the small percentage of steel, the rate of increase of deflection after cracking was very great so that these beams, due to excessive deflection and cracking, already became unserviceable at a small percentage of their ultimate load. However, at any stage they were very much *safer* than any of the beams reinforced with steel having a natural yield point.

If bars having a natural yield point are used with higher bond stresses because of their surface characteristics or high strength of concrete, mild steel quality is not sufficient.

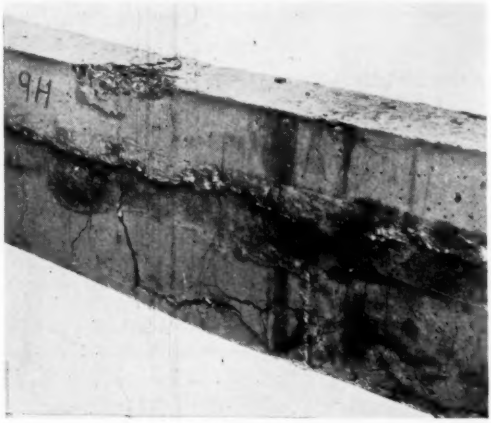

Fig. 17—Beam 9*H* after failure, 1-in. diameter American deformed bar twisted

Fig. 18—Beam 12H after failure, 1-in. diameter indented twisted bar

Higher bond stresses require higher quality steel so that stresses at failure are sufficiently high to allow the formation of wide cracks before the ultimate load is reached to indicate the approaching danger when the load-bearing capacity of the structure is not yet exhausted.

CONCLUSIONS

1. The tests have shown the superiority of deformed bars as against plain bars regarding the formation of cracks. The influence of the size of the bar, however, is much greater than that of its surface characteristics.

2. Whitney's theory is applicable to steel of any strength while the standard method is inadequate in assessing the load-bearing capacity of beams reinforced with very high strength steel. Even if the percentage is very low, the calculated concrete stresses at failure are excessive.

3. Cold worked bars of 1-in. diameter with a suitable surface pattern allow the utilization of at least 85 percent of their ultimate strength in concrete of $f_c' = 2500$ psi while the efficiency of bars having a natural yield point does not exceed 70 percent even in concrete of $f_c' = 5000$ psi.

4. Considering warning before failure, cold worked bars are superior to bars having a natural yield point.

5. Higher bond strength requires higher quality steel. If mild steel is used, improved bond, obtained by high quality concrete or by deformed bars, may lead to failure when a structure still appears to be safe.

6. The full economical advantage of higher strength steel can only be obtained if the idea of working stress is abandoned and the design is based on ultimate loads with reasonable safety factors.

ACKNOWLEDGMENT

The tests were carried out at R. H. Harry Stanger's Laboratory at Elstree, near London, in collaboration with Twisteel Reinforcement, Ltd., London, England.

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Discussion of a paper by K. Hajnal-Konyi:

Reinforcing Steel in Concrete and the Concept of Safety*

By EIVIND HOGNESTAD and AUTHOR

By EIVIND HOGNESTAD†

The author is to be complimented on adding new valuable information to our stock of test results, and on contributing significantly to a clarification of the philosophies involved in reinforced concrete design.

In discussing properties of beams reinforced with deformed bars, he has emphasized control of the spacing and width of flexural cracks in the concrete. The writer is in full agreement with such emphasis. Deformed bars meeting ASTM A305-50T are believed to exhibit such favorable bond characteristics that anchorage bond is no longer a serious problem in common designs. To keep down the spacing and thereby the width of flexural cracks is believed to be a much stricter test of bond performance than anchorage bond, particularly if allowable tensile stresses are increased from 20,000 psi to values approaching 30,000 psi. In the future development of deformed bars, it is felt that an emphasis of crack formation and a de-emphasis of pull-out tests is proper and necessary.

In terms of ultimate moments of beams failing in tension, the difference between Whitney's theory and other ultimate theories developed on the basis of test results, such as Bittner's, Brandtzaeg's, Jensen's and Saliger's, is small. The author compared his test results to Whitney's ultimate theory and found reasonable agreement. To this the writer only wishes to add that for tension failures such agreement must be expected with most of the many ultimate theories reported in the literature.

Ultimate load design, due to the nature of the problems involved, must be based on facts derived from experiments. As the author has pointed out, however, "it is important to consider also the way in which a structure fails." Such consideration enters the area of thought, of judgment, of design philosophy, and the writer finds himself unable to agree wholly with the views expressed by the author.

If consideration is limited to statically determinate members subject to static loads, the writer may go along with the view that, from the point of view of warning before failure, cold worked bars are superior to bars having a

*ACI JOURNAL, Mar. 1952, *Proc.* V. 48, p. 561. Disc. 48-37 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 24, No. 4, Dec. 1952. Part 2, *Proceedings* V. 48.

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natural yield point. If such statically determinate members may be called upon to absorb impact, however, it is felt that the outstanding energy absorbing capacity of members reinforced with bars having a natural yield point is more agreeable.

In multi-indeterminate structures, it is believed that the ductile beam behavior associated with a natural yield point is almost a necessity to develop the full ultimate strength and toughness of such structures through the formation of plastic hinges and the corresponding redistribution of moments. This is so even for static loading. If monolithic structures are subjected to impact loads, to seismic disturbances, or to blast effects, the great energy absorbing capacity resulting from the use of bars with a natural yield point is again desirable.

The writer is indeed in favor of efficient use of high strength reinforcement through proper ultimate load design. He believes, however, until convincing proof to the contrary is available, that ductile high strength steels produced with the aid of metallurgy are preferable to the relatively brittle steels resulting from cold working. It should be pointed out, on the other hand, that this is at present almost entirely a matter of opinion and judgment, as our knowledge of the inelastic behavior of monolithic reinforced concrete structures is only in its preliminary stages of development.

AUTHOR'S CLOSURE

Professor Hognestad has raised several relevant points. The author agrees with him that in the case of beams failing in tension the difference between Whitney's theory and several other ultimate load theories is small; Whitney's theory has, however, the advantage of greatest possible simplicity. It was interesting to find that within the whole range covered by the author's tests, *i.e.*, up to $M/bd^2c_p = 0.285$, the agreement between experimental and theoretical results based on Whitney's simplified assumptions was good and that the theory was applicable even for steel having an ultimate strength of 268,800 psi.

Professor Hognestad has expressed the view that in the case of impact and in statically indeterminate systems, steel having a natural high yield point is preferable to cold worked steel. As he has pointed out, this is for the time being almost entirely a matter of opinion.

Let us consider first the question of impact. The energy absorbing capacity of a structure may be assumed to depend mainly on the area enclosed by the load-deflection diagram. A structure of type (a), Fig. 13, is obviously unsuitable for resistance to impact. The following discussion is therefore limited to a comparison of types (b) and (c).

The author's tests were confined to static loading and Fig. 13 is related only to the maximum static load. However, if this load is reduced, as it generally happens in a testing machine (but not in a loaded structure), the load-deflection diagram can be continued with a descending line in both case (b) and case (c). Complete failure, *i.e.*, total collapse, occurs at a greater

deformation than that corresponding to the maximum static load and the load is smaller than the maximum. This descending part of the load-deflection line is most important for the assessment of resistance to impact but has not been shown in the diagrams.

If we exclude failure by diagonal tension or bond, there are two possible modes of "total failure."

(A) Crushing of the concrete which began at the maximum static load continues until the whole compression zone is destroyed.

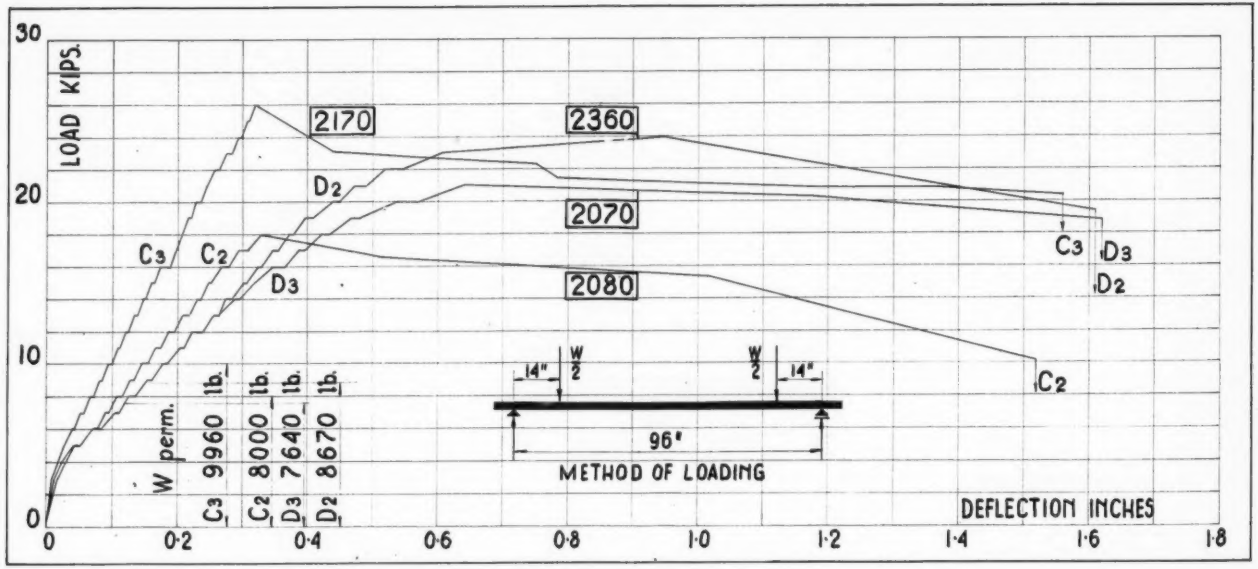
(B) The steel is fractured before complete destruction of the compression zone of the concrete has taken place. This occurs if maximum stress of the steel is reached at a stage when the concrete is still capable of sustaining an increased strain.

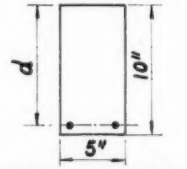
To make a structure as resistant to impact as possible, (B) must be avoided. Of the 38 beams described in the paper, 36 failed according to (A) and only the two beams, 20L and 20H which were reinforced with specially treated wire of high strength, failed according to (B). Nevertheless, the author is aware of the fact that failure according to (B) may occur with certain types of cold worked bars if they are used at relatively small percentages, while it is hardly possible with bars having a natural yield point. But it is possible to avoid failure according to (B) with cold worked bars if the following is borne in mind.

Most specifications require a minimum ultimate elongation of the steel, which is determined on a broken specimen of a specified gage length and is the result of the combination of a uniform elongation and necking. The contribution of necking to ultimate elongation is useless for shock absorption. It is not the total ultimate elongation, but the minimum value of the maximum uniform elongation which has to be specified. Provided that this value is not less than about 5 percent and the percentage of reinforcement is not unreasonably low, fracture of the steel will not occur. It is not difficult to guarantee a minimum uniform elongation of 5 percent if a controlled process of cold working is adopted.

To illustrate the preceding argument, Fig. A shows results of some recent tests in which an attempt was made to determine the descending part of the load-deflection line. Fig. A contains a few characteristic load-deflection lines. These have been continued beyond the maximum static load to "total" collapse which occurred by crushing of the whole concrete section. Beams of Series C were reinforced with plain round bars of a quality corresponding to steel of intermediate grade (*i.e.*, having a natural yield point of 40,000 to 50,000 psi); beams of Series D were reinforced with Tentor steel, which, however, had a different surface pattern from that of the bars shown in Fig. 2 (No. 17) and complied with ASTM A305-50T.

Contrary to the simplified diagram of Fig. 13b, lines C_2 and C_3 (Fig. A) show an immediate drop of the load on reaching the maximum. This occurred in both beams at the same deflection (0.32 in. = $l/300$). The diagram indicates the permissible load for each beam, worked out on the basis of a permissible stress of 20,000 psi for Series C and of 33,000 psi for Series D, with





CROSS-SECTION

Beam	Cube strength	Reinforcement				d	p	Area	Area/Wperm	Wmax
	psi	No	dia.	steel	sq.in.	in.	%	lb x in	lb×in/lb	W perm
C2	2080	2	1/2"	M.S.	0·397	8·87	0·895	21,100	2·638	2·25
C3	2170	2	9/16"	M.S.	0·494	8·84	1·120	31,700	3·183	2·61
D2	2360	2	·400"	Tentor	0·249	9·00	0·553	31,000	3·576	2·77
D3	2070	2	·380"	Tentor	0·226	8·80	0·513	28,625	3·747	2·75

Fig. A—Results of tests to determine descending portion of load-deflection line

$n = 15$, allowance having been made for the dead weight. For all four beams "total" collapse occurred at approximately the same deflection (between 1.52 and 1.62 in. = approximately $l/60$).

In a comparison of the areas enclosed by the load-deflection lines, the loads for which the beams were designed have to be taken into account. The area divided by the design load ("reduced" area) should give a fair basis of comparison. It may be seen from Fig. A that the area in question is much smaller for C_2 than for D_2 and D_3, although the permissible load for C_2 is greater than for D_3 and only 8 percent smaller than for D_2. The area for C_3 is slightly larger than for D_2 and D_3, but bearing in mind the greater permissible load for C_3, the comparison is again in favor of Series D. This can be recognized from the "reduced" areas tabulated in Fig. A.

A noticeable feature of Fig. A is the initial "advantage" of Series C (in the range of small deflections). This is due to the fact that in Series C steel of intermediate grade quality has been used, for which a working stress of 20,000 psi is permissible, with the consequence that the percentage of reinforcement is much higher than in Series D. If we substituted in Series C steel having a high natural yield point with a permissible stress of 30,000 or 33,000 psi, this initial advantage would be lost, and the comparison would be even more favorable for cold worked steel.

It would, of course, be premature to draw general conclusions from such a small number of specimens in low grade concrete, but Fig. A may be taken at least as an indication that the preceding theoretical argument is sound. Admittedly, the energy absorption capacity of steel *on its own* (*i.e.*, the area enclosed by the stress-strain diagram) is greater for steel having a natural yield point than for cold worked steel, but this energy absorption capacity can only partly be utilized if the steel acts as reinforcement of concrete since the concrete is crushed long before the maximum stress of the steel is reached unless the percentage of reinforcement is abnormally low. Consequently, if failure according to (B) is avoided and destruction of the concrete is the determining factor, there is no reason why a structure reinforced with steel having a natural high yield point should have a higher resistance to impact than a structure reinforced with cold worked steel.

Similar considerations apply to the redistribution of moments in statically indeterminate structures. The significance of plastic hinges is not the prevention of any increase of stresses at the hinge but the prevention of premature failure at a critical section until similarly high stresses have been reached at another section or at several other sections.

In this respect, some conclusions may be drawn from Fig. 6 and Fig. A. With steel having a natural yield point the deformation of the structure (whether statically determinate or indeterminate) at a critical section is comparatively small when the yield point is reached. By increasing the load further, the bending moment at the section where the steel has yielded does not increase any further, in fact it may decrease. On the other hand, with cold worked steel the increase of bending moment at the critical section does

Fig. A—Results of tests to determine descending portion of load-deflection line

not stop abruptly but is slowed down at an ever-increasing rate long before the maximum is reached.

This "slowing-down" should cause in statically indeterminate structures a similar redistribution of bending moments as yielding, the only difference being that the redistribution is more gradual. The hinge is "semi-plastic" in contrast to the "completely plastic" hinge formed by steel having a natural yield point. It would seem that the behavior of "semi-plastic" hinges is more favorable than that of "completely plastic" hinges and that the load-bearing and shock-absorbing capacity of comparable structures with "semi-plastic" hinges is at least equal, if not superior, to that of structures with "completely plastic" hinges, provided that fracture of the steel can be avoided and the stage reached when the whole concrete section is crushed. This is at present only theoretical reasoning and further research is necessary to prove this conclusively but the behavior of simply-supported beams points in this direction.

Title No. 48-38

Applications of Vacuum Concrete*

By K. P. BILLNER†

SYNOPSIS

Vacuum concrete processes were introduced in this country a number of years ago and their applications in building construction have been reported in the ACI Journal. During the last few years these processes have become known and adopted in many foreign countries, particularly in Europe, and this paper deals with these foreign developments, singling out new ways and methods in construction.

INTRODUCTION

Vacuum processing of concrete is the extraction of excess water from concrete immediately after it is placed and screeded to its required level. This is done by special apparatus which through use of a pump created vacuum draws off the excess water. This in turn permits the concrete to harden rapidly due to the reduction of the water-cement ratio.

The advantages of vacuum concrete are many and have been enumerated in this Journal‡ and elsewhere. However, to appreciate the universality of the vacuum process, it is necessary to review the many varied uses to which it has been put throughout the world. Recently, the author had the opportunity to observe some of the more outstanding examples of the use of vacuum concrete in foreign countries.

It is being applied in almost every type of construction particularly in conjunction with precast methods. Typical examples are precast vacuum processed water tanks, two-story apartments, housing units and canal revetments. Vacuum processing is being used extensively in Italy for making large size pipe to be subjected to high water pressures.

Each of the countries visited is discussed separately and the major projects, making use of vacuum processing and precasting, are described.

BELGIUM

Engineers for the City of Brussels, Belgium, recently completed construction of a rectangular concrete reservoir just outside the city.

Column footings and the floor were cast first. Next 80 columns 12 x 12-in. x 12 ft high, flared to 3 ft at the bottom and 18 in. at the top, were cast.

*Presented at the ACI 48th annual convention, Cincinnati, Ohio, Feb. 27, 1952. Title No. 48-38 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 7, Mar. 1952, *Proceedings* V. 48. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than July 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, President, Vacuum Concrete Corp., Philadelphia, Pa.

‡Lockhardt, William F., "Vacuum Concrete," ACI JOURNAL, Jan.-Feb. 1938, *Proc.* V. 34, p. 305.

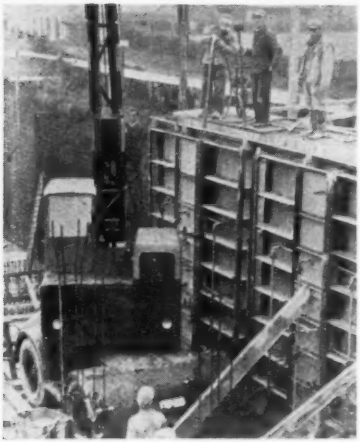


Fig. 1 (left)—Vacuum processing equipment in place on outside wall. Note lack of usual form work

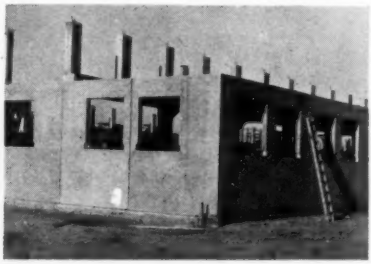


Fig. 2—Sandwich-type precast walls used on industrial plants in Canada

The columns were cast with only two vacuum-lined steel forms held to the floor by vacuum. External vibration was by a very small vibrator affixed to the forms by vacuum. The forms were released half an hour after the concrete was placed and were used five times each day.

The 12-ft outside walls were cast similarly (Fig. 1). Ribbed roof slabs were cast in concrete molds, vacuum processed, and then lifted and set in place on the columns. The roof was designed for a live load plus a dead load of 500 psf due to the 3 ft of earth insulation that it carries.

Tie wires attached to the top of the outside walls and run between the precast roof slabs to the opposite walls were prestressed by the Magnel-Blaton system. Two heavy wires, with a 12 ft center to center spacing, were used for these ties.

A. Achten is chief engineer for the Compagne Intercommunale Bruxelloise des Eaux, general contractor for the reservoir. The vacuum work was done under supervision of Jacques Arts, engineer for Syndicat Vacuum Concrete in Brussels.

CANADA

Creaghan and Archibald, Ltd., Montreal, have specialized in vacuum concrete, sandwich-type precast walls and roofs. The wall panels generally span the distance between columns and consist of two reinforced vacuum concrete slabs enclosing a 2-in. layer of foam glass (Fig. 2). The two slabs are cross reinforced to allow for horizontal shear and thus act as one slab. They have approximately the full strength of a 6-in. concrete wall and approximate the insulation of a 24-in. brick wall. Because of the excellent insulation, economy and speed of erection they afford, walls and roofs of this type have been erected in about half a dozen large plants in Canada, including the International Paper Company at Gatineau, Quebec, and the

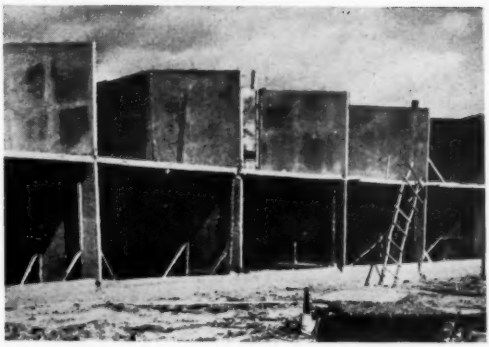


Fig. 3—Precast walls used in two story housing project in Bogota, Colombia

new Columbia Cellulose Plant at Watson Island, British Columbia. These buildings are not only structurally sound but pleasing in appearance.

COLOMBIA

Vacuum Concrete de Colombia, Alvaro Ortega, general manager, is erecting large housing projects characterized by an unprecedented economy of materials. The walls of these buildings are precast and are only 2½ in. thick. The new type of roofs (known as the Ortega-Billner roofs) are curved and cast on top of one another and vacuum processed. They are separated by a vacuum lifter which is also used for erecting the roofs and walls. Fig. 3 shows erected walls for two-story houses in serial construction and Fig. 4 a roof being set in place, using the vacuum lifter.

The original plans for these houses were by Paul Lester Wiener and Jose Luis Sert of New York, consultants on the new city plan for Bogota.

EGYPT

The first precast multistory apartment project, 730 flats and 42 shops

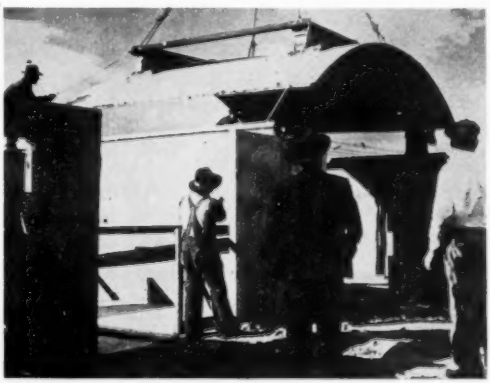


Fig. 4—Ortega-Billner roof being lowered into place by vacuum lifter

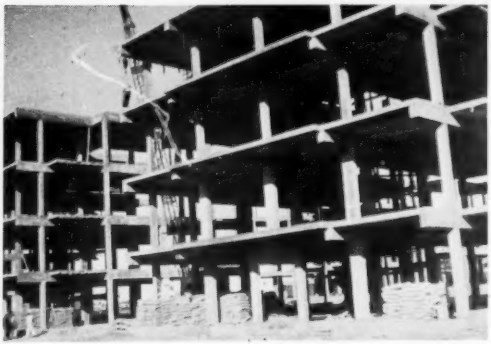


Fig. 5—Structural frame of precast members used for apartment building in a suburb of Cairo, Egypt

comprising 63 separate building units, has been erected at Heliopolis, suburb of Cairo. The author conceived a preliminary design consisting of hollow columns with trough-shaped cross section main beams, and a new type of hollow slab economical of materials. This design was unique in multistory precast apartment buildings. However, due to the inexperience of Egyptian labor, it was decided that the columns and beams must be simplified even at a sacrifice of materials and time.

Therefore, the columns were solid with a conventional male socket at the base and with the reinforcement protruding about 9 in. above the top end, which enabled the beams and columns to be tied together at erection. The beams were precast to three-fifths the design thickness with reinforcement protruding 4½ in. above the upper surface. The ends of the beams were so designed that they surrounded the protruding vertical reinforcement of the columns, thus forming a mold for "tying-in" cast-in-place concrete.

After the columns and beams for the first story were erected, the precast hollow floor slabs were placed. These floor slabs became the edge forms for placing the upper sections of the beams. These sections, with their protruding reinforcement, were then filled to the level of the slabs. With this tying in

Fig. 6—Finished appearance of buildings shown in Fig. 5

of the column and beam reinforcements the entire structure had the strength of monolithic concrete. Fig. 5 shows the structural frames of some of the apartments and Fig. 6 some of the finished buildings.

There was relatively little variation in column sizes, therefore it was possible to precast them horizontally in gang molds. Bottom molds were used only on the first layer of each group; thereafter the edge forms were stripped, raised and secured to the member itself which, after oiling, served as the bottom mold.

The beams, being of varying lengths and widths, were cast side by side on simple wooden pallets. Edge forms of thin sheet steel panels stiffened by wood cross members were used on one side only. The previous day's pour, usually about 300 lineal feet for each group, served as an edge form for the opposite side. Wooden bulkheads formed the ends.

Vacuum processing of the columns and beams was considered but again, due to the inexperience of local labor, it was decided to vacuum process the floor slabs only. However, they make up about 75 percent of the total precasting. The slabs were generally 4 ft wide by 8 in. thick, with an average length of 15 ft. They were hollow and the air space between them comprised 60 percent of the total volume. Core forms were 18 x 5¾ in. in cross section. Therefore the top slab over the core had an average thickness of 1½ in. and the bottom slab was only ¾ in. thick. In casting, the bottom layer of concrete was placed first, then the core forms, and finally the rest of the concrete. A vacuum processing mat was placed on top of the fresh concrete and vacuum was applied for 12 to 15 minutes. The size of the core forms was reduced by vacuum applied through the center and then pulled out. Edge forms were removed at the same time. The mat was then removed and placed on the adjoining slab. Thus the same edge forms and cores could be used about 10 times each day.

Cement factor for the floor slabs was 300 kg per cu m (505.7 lb per cu yd) of concrete, but after processing they showed strengths corresponding to unprocessed slabs having 400 kg of cement per cu m (674.3 lb per cu yd) of concrete.

This method of construction was a complete innovation in Egypt. Nevertheless, the contractor found it possible every 3 months to cast and erect the complete skeletons of seven 4-story buildings, with approximately 400 sq m (4304 sq ft) per floor every three months. Near the end of the project this was accomplished in 2½ months. The savings in material and form lumber make this an outstanding example of what can be done in precasting multistory buildings.

The owner is the Cairo Electric Railways and Heliopolis Oasis Co. The African Enterprise and Development Co. was the general contractor, and the vacuum concrete phases of the job were under the supervision of George V. Carlson of Billner Vacuum Concrete, S. A.

Fig. 7 — Vacuum processing equipment in place on revetment mats to be used for lining the Ottmarsheim Canal in France

FRANCE

Electricite de France, a branch of the French Government, has just completed lining the banks of one portion of the Ottmarsheim Canal on the Rhine River development with precast vacuum concrete slabs. This work comprised 6,000,000 sq ft and there are two more sections to be done in the same manner. Each slab is approximately 30 ft long, 8 ft wide, and 3½ in. thick. Slabs were cast on a concrete platform, vacuum processed, and lifted off the platform by vacuum lifters the morning after they were cast. In placing them, a space approximately 2 in. wide was left between the slabs; hooked steel protruded so that when these grooves were filled the slabs were tied together. Fig. 7 shows a vacuum processing mat and Fig. 8 the casting yard.

Another project of the same magnitude using articulated slabs has been started on the Rhone River development. Slabs are vacuum processed to increase their strength, and are handled by a new improved type of lifter, which has been named the "Octopus." It is designed so that it can grab automatically upon touching the surface of objects of almost any shape, whether they have straight or curved sides. Fig. 9 shows the Octopus lifting articulated concrete revetting mats off the casting bed. It can pick up mats consisting of twenty individual pieces with a total length of 25 ft and width of 4 ft weighing two tons, and stack them in approximately 60 seconds.

Among the many items made in France by the vacuum process are electric

Fig. 8—General view of the casting yard for precasting revetment slabs

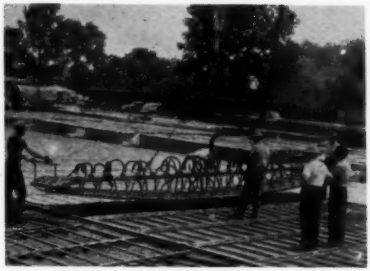


Fig. 9—The Octopus vacuum lifter in operation

poles. They are generally made in lengths of 35 to 50 ft with vacuum processing through the sides and bottom of the steel forms in which they are cast. The steel side forms with the fresh concrete are lifted, and the entire pole, in which the concrete has not yet set, is placed on a sand bed and the side forms removed in about 20 minutes. When only two days old, these poles can be picked up at one point (Fig. 10).

The developments in France are sponsored by the Societe du Vacuum Concrete, I. Leviant, manager, with headquarters in Paris.

ITALY

Professor Vacchelli, University of Milan, has made a series of tests of vacuum concrete which confirm previous tests made in the United States, France and other countries.

Societa Italiana del Vacuum Concrete, Tullio Delfini, manager, Milan, is precasting 3,000,000 sq ft of lining for pressure tunnels for works at Talvera, Avisio and Adige.

Similar slabs are precast for the outside facing of the electric stations at Predazzo and Tavazzano. These slabs are approximately 5 x 2 ft, and 1¼ in. thick and are reinforced with a light wire mesh. They have a remarkable strength of about 9000 psi and an absolutely smooth wearing surface. The plants for making these slabs are at Predazzo and Mori near Trento. The ornamental slabs for buildings have a white cement surface. Work is now in progress to precast flumes for irrigation projects. Fig. 11 shows a power station faced with precast slabs.

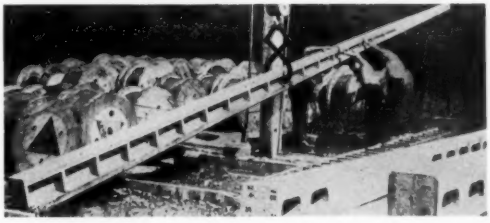


Fig. 10—Precast utility pole being lifted only two days after casting

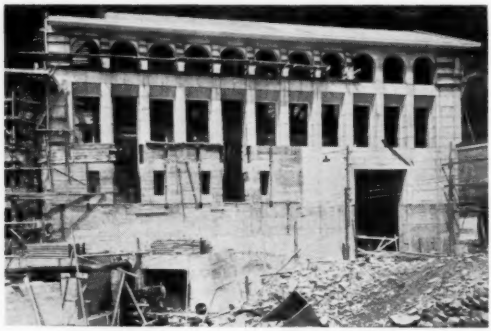


Fig. 11—Power station in Italy faced with precast slabs

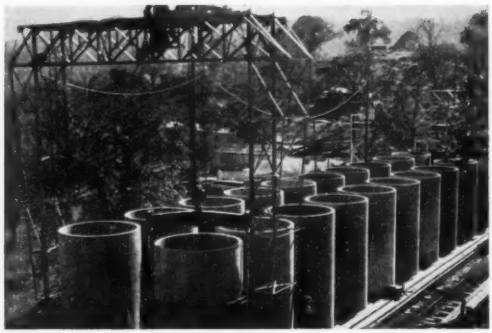


Fig 12—General view of pipe precasting yard at Mucone, Italy

Mantelli and Ci., Ferrocemento, Rome, F. Piccinini, manager, are making 100,000 tons of large diameter pipe of two types. At Mucone, southeast of Naples, 10-ft diameter pipe are cast in lengths of 16 ft 8 in. The wall thickness is 4½ in. and the strength of the concrete averages 7000 psi for a 1:5 mix. Both inside and outside vacuum forms are removed half an hour after casting the concrete and the pipes are lifted off the casting platform by the vacuum lifter the following morning. Then, after required strength is obtained, they are turned to a horizontal position by a special vacuum lifter and wound with high strength wire for prestressing. These pipes have no steel membrane and yet they withstand a pressure of 12 atmospheres, *i.e.*, a 400-ft head of water, without any moisture coming through. An outside protective coating for the prestressed wire, 1 in. thick, is placed and vibrated and then vacuum processed. Fig. 12 shows a general view of the yard, Fig. 13 the placing of concrete in a form.

The pipe made at the Volturno Plant has an inside diameter of 14 ft and a wall thickness of 12 in. Each section is 15 ft long and weighs 52 tons. They are reinforced in the ordinary manner (not prestressed). Inside and outside forms are removed 30 minutes after the concrete is placed and the pipes are

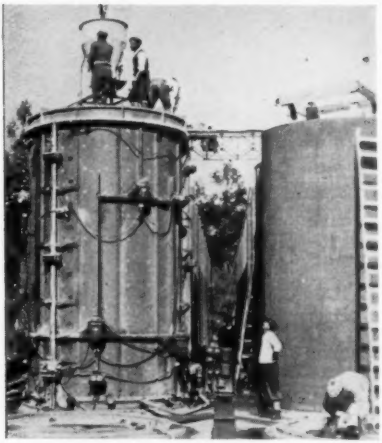


Fig. 13—Placing concrete in a vacuum process form for pipes

lifted off the casting bed by a vacuum lifter the morning after they are cast. The pump requires only 15 horsepower for this heavy lift. The mix is 1:7 and the pipe is to withstand a 60 ft head of water pressure.

SPAIN

The most interesting work is a contract for unreinforced large diameter pipe. The need for saving steel at this time is universal but is particularly acute in Spain. Therefore, it was decided to use very high strength concrete and to support the pipe in a cast-in-place trough-shaped lean mix concrete bed. The average diameter of these pipes is 5 ft; the wall thickness is 2⅜ in. The work is sponsored by the Spanish Vacuum Concrete Company, Madrid,

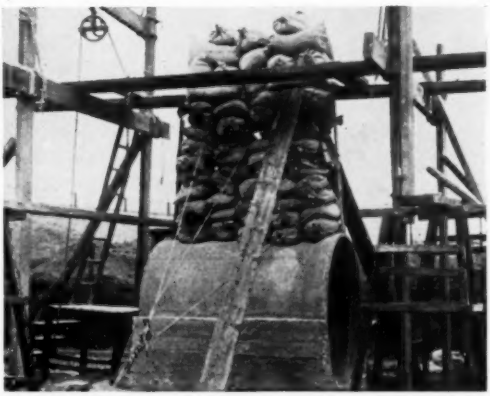


Fig. 14—Test load of unreinforced pipe made in Spain

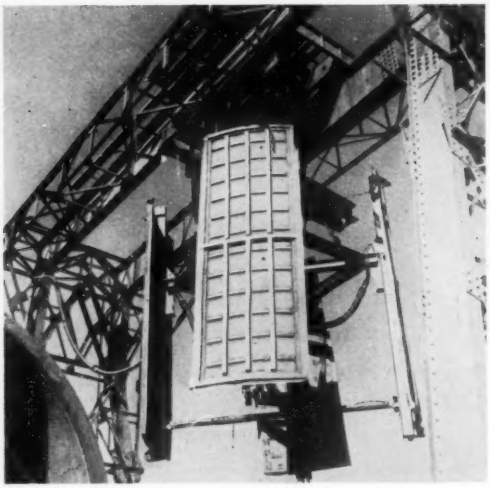


Fig. 15—Vacuum lifter for 52 ton pipe

of which Alfredo Semelas is manager. Fig. 14 shows test loading of this unreinforced pipe and Fig. 15 the vacuum lifter.

UNION OF SOUTH AFRICA

The vacuum concrete processes are rapidly coming into use. They are sponsored by Vacuum Concrete Pty., Ltd., Africa, Peter J. Doanides, manager. One of the outstanding jobs is a partition wall in the Van Dyk Gold Mine. The mine is 4800 ft deep, and the 6-in. thick 15-ft wide partition wall was built in 40 days (continuous 24-hour operation), using only one 10 ft high vacuum form, which was moved up every two hours for the next pour. The wall was built to eliminate air currents retarding operation of hoists traveling at 3000 ft per minute.

Vacuum concrete construction is also being used for native housing and for streamlined precast concrete struts in the mine shafts.*

One of Doanides' most recent developments is precasting of large circular tanks. Depending on the diameter of the tank, 36 to 48 precast panels the full height of the tank are used. The vacuum concrete process makes possible economical use of rib panels cast in concrete molds. The panels are vacuum processed from the sides and top. The early strength obtained and the use of the vacuum lifter makes possible the lifting of the sections from the molds each morning and the use of the same molds once every day. In this way, with one set of side forms and six molds, it is possible to cast all required sections for a 200,000 gallon tank in five to eight days. Costs are greatly reduced in industrial plants requiring a number of tanks, where it is possible

*Doanides, Peter J., "Streamlined Concrete Buntons for Mine Shafts," ACI JOURNAL, Dec. 1951, *Proc.* V. 48, p. 309.

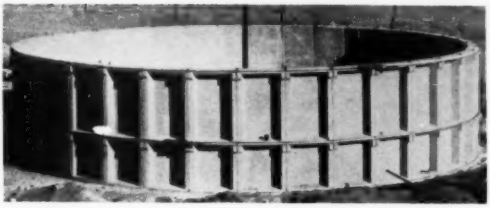


Fig. 16—Precast water tank in the Union of South Africa

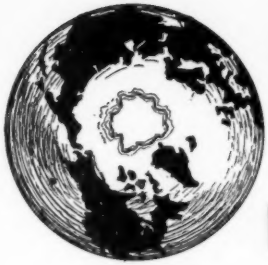
to establish a small casting yard and produce enough precast sections for a 100,000 gallon tank every two days. The precast panels are held together by post-stressed wire ropes tightened at the bottom, at the lower third and at the outside top of the walls.

Two x 1-in. sponge rubber strips are fixed either side of grooves in the vertical ribs of the precast panels. These sponge rubber strips make possible the use of the vertical grooves as vacuum holders during the erection of the tank sections, and provide temporary watertightness of the tank before concreting the grooves. The grooves are filled with concrete after the tank has been filled with water. Thus, the post-stressed cables will have their maximum working stress when these jointing grooves have been filled with concrete. With the well-known watertightness obtained with vacuum processed concrete, a thickness of 3 in. for the precast panels has proved adequate. The total quantity of concrete is thus greatly reduced compared with tank walls cast *in situ*, and the danger of shrinkage cracks is eliminated. Vacuum holders are used for adjusting the precast sections during the course of erection.

Total erection time for the tank shown in Fig. 16 was 8 hours and 40 minutes. Doanides claims that precasting walls of circular tanks can reduce construction time to a fraction of that required by other methods. During excavation and pouring of the foundation ring, the precast sections are cast and cured. Upon completion of the foundations, the wall panels are ready for erection, and the tank can be ready for use within two days.

SUMMARY

This report indicates the rapidly growing interest in precasting methods abroad. The reason is that it saves form lumber, steel and cement. Due to its inherent better control, uniformly high strength concrete can be had. This in turn means lighter sections and less construction time.



Letters from Readers*

BY WAY OF SYNOPSIS

EDWIN ROSE, H. J. GILKEY, RAYMOND C. REESE and ARTHUR P. CLARK discuss bond values for reinforcing bars in the 1951 Building Code.

BRUCE E. CLARK presents charts for the field solution of grout mix problems.

Why Classify Old-Style Deformed Bars as Plain Bars in the 1951 Building Code? (LR 48-24)

Although the old-style deformed bars referred to in this discussion have pretty well gone out of existence, making the question of a change in the Code an academic one, the several answers furnished by Members thoroughly familiar with the development of the 1951 Building Code give an excellent insight into Standards work in the Institute.

On the whole, the revised "Building Code Requirements for Reinforced Concrete (ACI 318-51) is an advancement in efficient and economical design and the committee is to be commended on its fine work.

However, I find it difficult to justify the use of a lower allowable bond stress for all the old types of deformed bars (by considering them plain bars) than the values formerly used. For years all types of structures have been designed using old code values and have given satisfactory service with no failures by bond. A question arises as to why it was suddenly necessary to reduce allowable bond when the former design value had proved satisfactory for many years.

It may be stated that we are not concerned with the older type of deformed bar; that we have only the new type deformed bar which all mills are now rolling. Assuming that this is true, there is a transition period when some jobs use old-type deformed bars from old stocks. Even if this occurred in only a small number of jobs, some provision should have been made for this transition period.

To try to follow the committee's reasoning behind this change regarding classing old deformed bars as plain bars, I have reviewed some of the literature on the subject. The only published material I have found that seems to partly substantiate such a change is by Arthur P. Clark† in 1949. In this paper,

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 7, Mar. 1952, *Proceedings*, V. 48. Separate prints are available at 35 cents each. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Clark, Arthur P., "Bond of Concrete Reinforcing Bars," ACI JOURNAL, Nov. 1949, *Proc.* V. 46, pp. 161-184.

tests of the bar designated No. 2 indicate values of bond not much better than the plain bar which was also tested in this same series. It is the only one of the older type deformed bars, of which several others were tested, that show values near those of the plain bar. An inspection of the picture of the bar shows only slight deformations spaced at what appears about 1½ bar diameters, and gives appearance of having such poor deformations so as to really appear little better than a plain bar. All other older deformed bars seem to give considerably better results.

Why could not bond for the old type of deformed bar have been provided for as follows:

1. For the old deformed bar, use the 1947 Code value for bond, that is, $0.05 f_c'$ for beams and slabs.

2. Class the particular type of old deformed bar which has such small deformations spaced so far apart (as the No. 2 bar in Mr. Clark's tests), as a plain bar. A simple specification would have defined the minimum requirements for the old type of deformed bar.

In the interest of clarification and simplified detailing during the transition period, I believe the Code should have included some provision for the old type deformed bar other than being considered as a plain bar.

EDWIN ROSE, Structural Engineer, Bureau of Reclamation, Denver, Colo.

The points raised by Mr. Rose, and the suggestions he offers are about what might be expected from an alert, thoughtful person not fully conversant with the background of recent ACI Code changes. Actually the A 305 bars are the result of a widely publicized incubation period extending over some eight years, during which manufacturer after manufacturer has converted until now each of the 29 leading bar producers is rolling one of the six patterns meeting ASTM A 305 requirements. In other words, old-type bars were virtually off the market prior to 1951 and the February 1951 changes in "Building Code Requirements for Reinforced Concrete" constitute little more than confirmation of accomplished fact.

A unique angle of the new-bar development is the interesting fact that after some forty years of lethargy the steel interests themselves took the lead in pushing for the development and adoption of better bars. In fact over a period of four or five years I have, as Chairman of both ACI Committee 208 on Bond Stress, and of the Standards Committee, borne the brunt of considerable impatience on the part of the steel bar associations and individual manufacturers of bars, over the slowness with which the "orderly processes of ACI Standardization" progressed. It is at least novel and mildly comforting to be accused by implication not of slowness, but rather of excess haste. With the initiative and responsibility for these code changes so heavily vested in the producers the problem of left-over stocks of old-type bars didn't exist, but if it had it was literally "within the family" and not something for serious consumer concern.

Other aspects of Mr. Rose's comments, along with some reiteration of the foregoing, will be covered in an attempt to review briefly the background of the 1951 Building Code changes. This review is offered mainly as a series of unsupported statements.

1. Committee 318, Building Code, adopted exactly the recommendation formulated in February 1949 by Committee 208, Bond Stress.

2. There have been in the past innumerable patterns and types of deformed bars of which some 15 or 20 were current during the past decade. These patterns had quite a range of excellence, some fairly good, others poor. A deformed bar—good, bad or indifferent—has always been a *deformed bar* without definition or distinction. The ASTM A 305 tentative specification represents the first attempt to define such a bar in terms susceptible of measurement. It constitutes a studied attempt to capitalize upon some

things we have long known and upon recent added information and activities culminating in the development of patterns for greatly improved deformed bars. There was and is no existing basis for classifying some bars as better than others without having gone through the same exhaustive test procedure that led to the A 305 specification.*

3. As long ago as 1913 such load-slip curves as those of Fig. 23 and 24 of Abram's University of Illinois *Bulletin* 71 show that within the very small amount of slip that can be tolerated in a beam without serious cracking only a few of the very best patterns of the then-current deformed bars showed an effectiveness even slightly superior to that of plain bars. Over the years there was little progress toward better. Within the workable-slip range, the moderate code advantage long accorded to deformed bars over plain has been largely fictitious and based more upon the slowing-down of final collapse (if the bar didn't split out) than to any effective contribution to *workable* bond.

4. To say that an absence of failures indicates adequacy of design is a common but highly questionable type of statement. We don't expect the structures we design to fail under normal usage. Until we get an *actual* failure we don't know how far we may have intruded upon the margin of added resistance we endeavored to supply by the so-called factor of safety. Even after we get a failure of a structure, only rarely have we a basis for knowing just what constituted the primary or initial breakdown which functioned as the trigger to collapse. With how many failures of reinforced concrete structures is Mr. Rose personally familiar and, if any, does he know the primary cause of the failure? On the other hand if the reserve strength under normal use is but 25 percent in bond against 50 percent against failure by tension in the steel, is that a balanced or fully satisfying design?

With an increasing tendency to "up" the stresses in the steel the relative weakness in bond and diagonal tension (and the two usually fail together) has been becoming more pronounced. No, I cannot concede that at the present stage of engineering design, mere absence of failure is evidence of adequacy.

5. The work of Menzel, supported by the extensive researches of Clark have strikingly shown the great reduction of bond for top rods due to settlement of the concrete and water gain beneath the bars. The deep-cut corrugations of the A 305 bars make them greatly superior to any of the old-type, although even for the A 305, there is still pronounced top-rod reduction in bond.

6. As mentioned initially, the discussion implies that the change-over has been so sudden as justifiably to leave appreciable stocks of old-type bars on the market. This is not the case. Only a very stubborn or a very sleepy producer or dealer could have been pinched by the change-over. In February 1949, Committee 208 released its recommendation which was published in *Engineering News-Record*, Mar. 3, 1949, and in the ACI JOURNAL *News Letter*, Apr. 1949, and ACI JOURNAL, May 1950. The Rail Steel Bar Assn., The Concrete Reinforcing Steel Institute and the American Iron and Steel Institute have published voluminously, heralding the new type ASTM A 305 bars. The leading bar producers have been extremely impatient of the ACI's Bond Committee (208), its Building Code Committee (318) and the Standards Committee procedures which delayed for by about two years the activation of a recommendation widely publicized and known to be coming. The A 305 bars have had a long transition period; they had already largely crowded old-type deformed bars out of the picture more than a year before the ACI 1951 Code changes became effective. Laws drastically affecting income taxes, availability of materials, appropriations for enormous undertakings are passed (or directives are handed down) with a fraction of the warning given in the present instance. It was the consensus of the Committee and of the producing steel companies that, once the change-over to the improved bars was decided upon, the faster the transition could be made the better. It was deemed important to provide every producer with an economic incentive to put himself in line at the earliest possible moment.

7. Much as we ACIers might like to have it so, the ACI Code is not yet the law of the land. Should there be any organization so

*Report of ACI Committee 208, Bond Stress, "Proposed Test Procedure to Determine Relative Bond Value of Reinforcing Bars," ACI JOURNAL, Feb. 1945, *Proc.* V. 41, pp. 273-292; Nov. 1945 Supplement, pp.292-1—292-8.

backward as still to find itself stocked with old-type deformed bars, it is operating in an orbit where the ACI or any other up-to-date code isn't likely to be of consequence or even known.

Closing comment

To anyone who has functioned on a technical committee or a code-evolving body it is axiomatic that not all of the provisions of any group report can be supported either by facts or by logic. Even where there is a solid backlog of performance or research data, there are still differences of interpretation and differences in the attempts at extrapolation to make the findings applicable to other specialized and generalized situations. Code provisions not directly tied to experiment or specific experience are even less sacred and are, more often than not, the result of such factors as: mere committee consensus, an averaging or compromising of divergent views, some member's "expert" opinion, or an offhand statement that "got by" unchallenged. Without belittling the importance of our technical committees, for they constitute the best method yet devised for pooling and codifying collective experience, the fact remains that close familiarity with committee affairs tends to reduce the halo to a hat band—sometimes a soiled one. A more general realization that committee feet are always embedded in clay would entail some loss of awe and prestige but it should beget more and better discussions from the laity. Even though I have disagreed with Mr. Rose's contention and recommendation, I maintain that he is to be commended for offering them, especially since he may well have been functioning as an unwitting spokesman for others of the Institute's "readership." It is to be hoped, moreover, that other ACI members will likewise be as constructively critical of committee offerings as they would be of those from an individual. It takes questions and suggestions to beget better answers and to spark improvements.

H. J. GILKEY, Professor and Head, Dept. of Theoretical and Applied Mechanics, Iowa State College, Ames, Iowa. Chairman, ACI Committee 208, Bond Stress

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In answer to Mr. Rose's question as to why ACI Committee 318, in revising the "Building Code Requirements for Reinforced Concrete (ACI 318-51)," did not see fit to have three categories of bars, *i.e.*, those meeting ASTM A 305-50T, those which are deformed but do not meet the requirements, and plain round bars, the following brief thoughts are offered:

1. The new style bar has a definite specification, easily verified, and the stresses can be correlated with the bars.

2. The old style bars had no specific deformations. They varied according to the rolling mill's practice or whim. It would, therefore, be complicated to determine a lesser bond stress that would be applicable to such a wide assortment of deformations.

3. It was stated as the unanimous opinion of the rolling mills that after the middle of 1951, only new style bars would be rolled. The current stringent steel situation has made it impossible to meet this statement 100 percent, and yet it is close to fulfillment.

4. Therefore, the Committee felt they were justified in dividing bars into two classes—those which do conform and those which do not conform.

5. Since the recommendations of ACI 318-51 become effective only when ratified by the proper legal authorities (legislature, city councils, or what-not) and since time is involved to make such specifications legally effective, it was felt that the stock of old-style deformed bars would be pretty well consumed by the time ACI 318-51 was legally applicable and that the designer, in conjunction with the local authorities, would probably have some leeway in the interim period to work under either the old or the new code, depending upon the bars available.

6. Finally, it was considered unwise to perpetuate any thought of a "half-deformed" bar, even for a brief period.

RAYMOND C. REESE, Consulting Engineer, Toledo, Ohio

In reference to the discussion by Mr. Rose, I would comment that there were many old-type deformed bars and the bond resistance developed varied from just a little better than plain bars to something less than that developed by the new types. However, in some cases premature failure was due to splitting action. Code and specification requirements relating to deformed bars were obviously improper with the wide range and questionable bond values of the old deformed bars.

To correct conditions it was necessary first to have a definition for a deformed bar. After extensive tests it was possible to outline the minimum requirements for the surface deformations which would justify the use of a specified bond stress. Inasmuch as the producers indicated they could and would provide bars to meet the minimum requirements, later incorporated in ASTM A 305, there was no need for another specification for what might be called a semi-deformed bar.

All of the principal producers are now exclusively rolling the new types of bars and considering the shortage which has existed for some time there can be little tonnage left of accumulated stocks and what there may be would certainly be used up before a new provision of the ACI Building Code could be formally adopted.

ARTHUR P. CLARK, Research Associate, Fellowship of Committee on Reinforced Concrete Research, American Iron and Steel Institute at National Bureau of Standards, Washington, D. C.

Grout Mix Computation Charts (LR 48-25)

During grouting operations it is frequently necessary to change the consistency of the grout in the mixer, sump or supply line by the addition of either water or cement. It is the responsibility of the grouting inspector to make or check these determinations.

While analytical solutions are not difficult, they are time consuming. The accompanying charts have been devised as an aid to the grouting inspector in making rapid determinations of the proper proportions of water and cement necessary to provide grout of any desired consistency, including changing a given amount of grout of known consistency to a different water-cement ratio. Examples illustrating the use of the charts are shown below each chart.

The charts are prepared on the basis of the following relationships:

$$\text{Water-cement ratio, } w/c = \frac{\text{cu ft of water}}{\text{cu ft (sacks) of cement}}$$

$$\text{Volume of mixed grout} = w + \frac{c}{2}$$

From the above relationships, the following formulas can be derived:

To thin mixes—

$$W = \frac{N(B - A)}{A + 0.5}$$

To thicken mixes—

$$C = \frac{N(A - B)}{B(A + 0.5)}$$

Number of sacks of cement in a given quantity of grout—

$$S = \frac{2N}{1 + 2A}$$

Where:

C = cement required to thicken mixes
N = cu ft of grout on hand
A = initial w/c
B = desired w/c
W = water required to thin mixes
S = sacks cement in a given quantity of grout

The above formulas are sufficient for solving grout mix problems. However in the field the charts solve the problems more quickly, and with less chance of mistake. For field computation of grout mixes and changes, water-cement ratio = cu ft of water divided by sacks of cement. To use mix thinning and thickening charts lay a straightedge from intersection of initial and desired water-cement ratios to lower left-hand corner of chart (point x). Read sacks cement or cu ft of water corresponding to cu ft of grout.

BRUCE E. CLARK, Geology Section, Sacramento District, Corps of Engineers

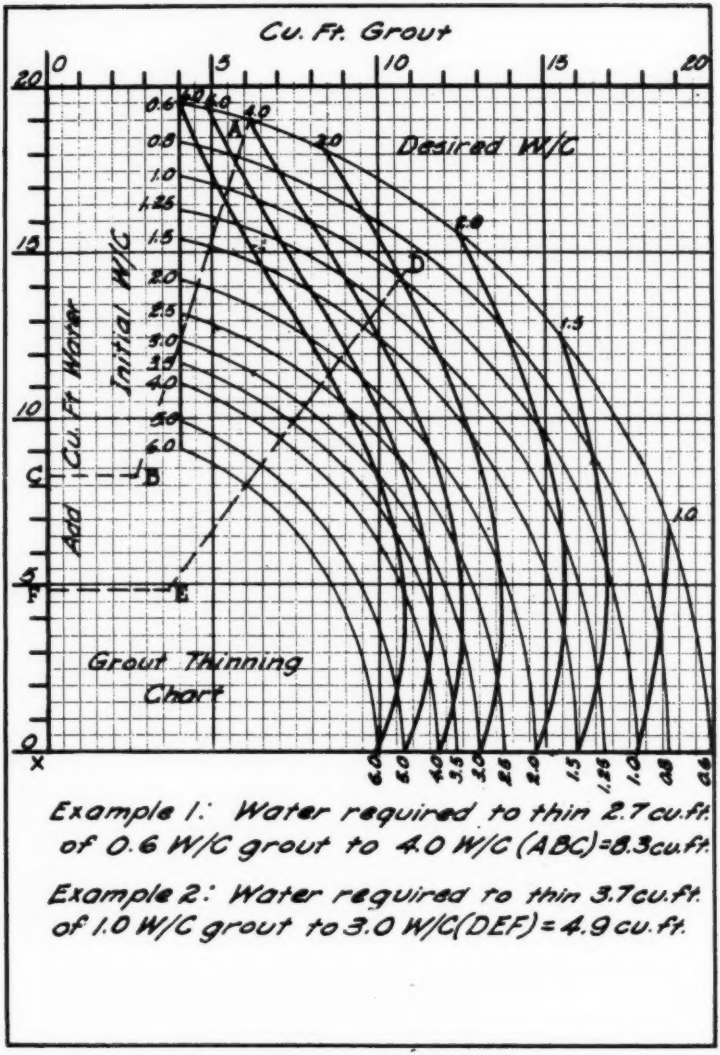
Cu. Ft. Grout
Desired W/C
Initial W/C
Add Cu. Ft. Water
Grout Thinning Chart
Example 1: Water required to thin 2.7 cu.ft. of 0.6 W/C grout to 4.0 W/C (ABC) = 8.3 cu.ft.
Example 2: Water required to thin 3.7 cu.ft. of 1.0 W/C grout to 3.0 W/C (DEF) = 4.9 cu.ft.

Fig. 1—Chart and examples for thinning grout

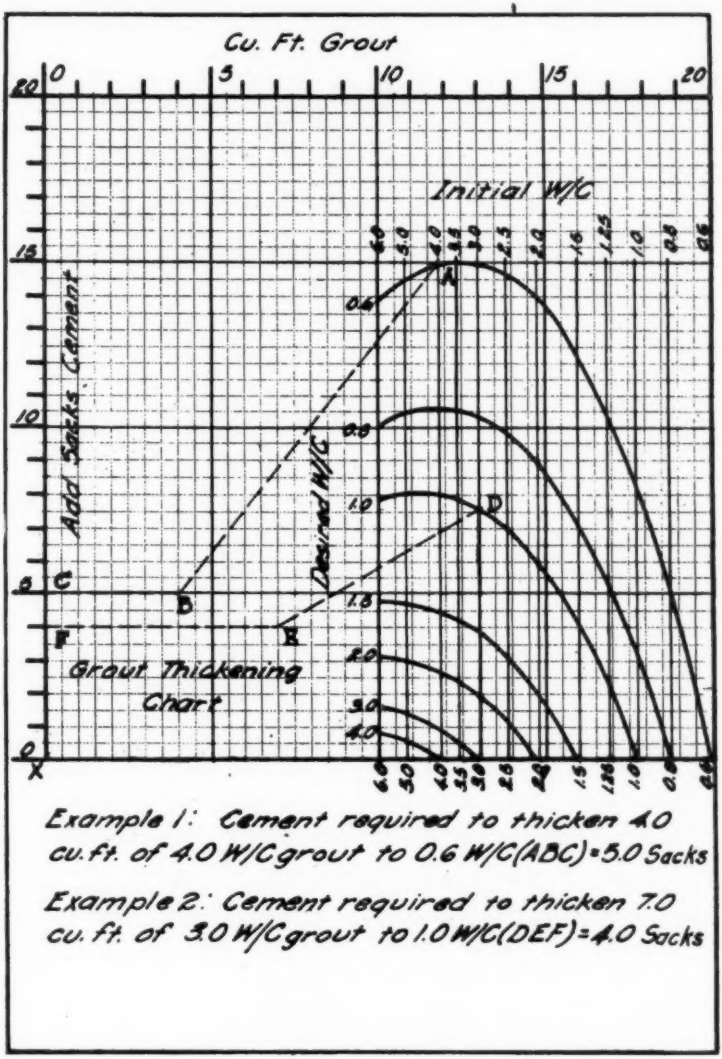

Fig. 2—Chart and examples for thickening grout

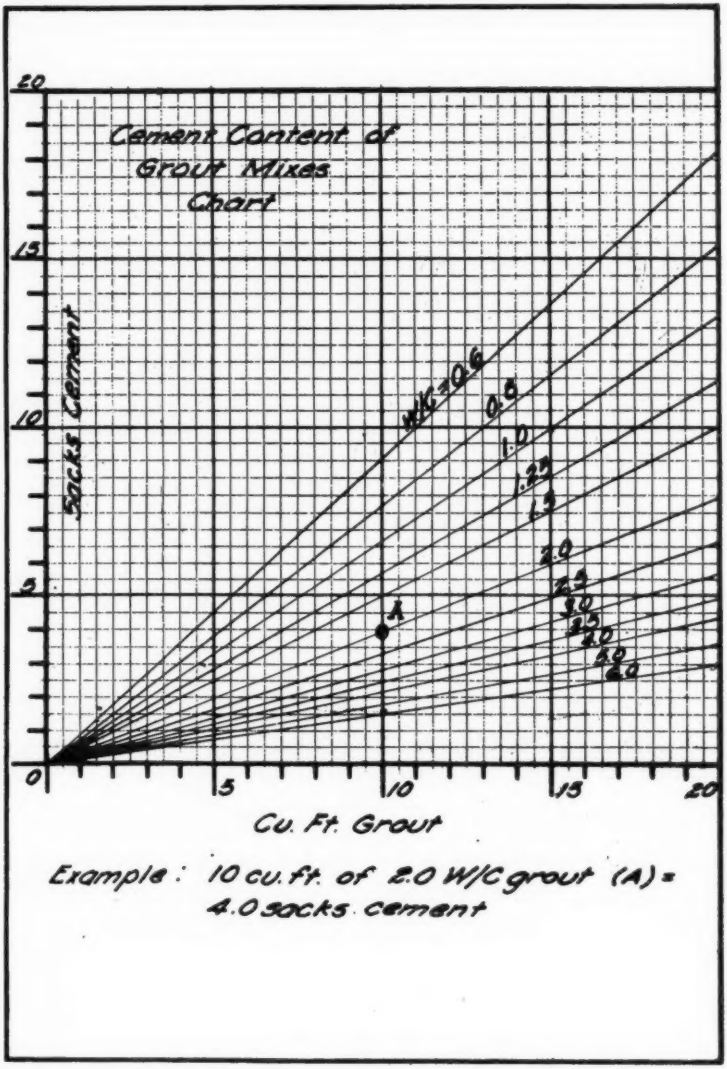

Fig. 3—Chart for determining amount of cement in grout

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Cleaning and sealing joints in concrete roads

Concrete and Constructional Engineering (London), V. 46, No. 11, Nov. 1951, pp. 346-348

Reviewed by GLENN MURPHY

Describes British and American equipment as demonstrated by the Road Research Laboratory.

Transporting and storing loose cement

Concrete and Constructional Engineering, V. 46, No. 11, Nov. 1951, pp. 337-345

Reviewed by GLENN MURPHY

This article consists of a description with drawings and photographs of methods evolved for handling loose cement. Primarily British and continental practices are described.

Measuring the prestress in steel wires in concrete with a wire stress meter (Messen der Vorspannung in Drahten bei Stahlsaitenbeton mittels Drahtspannungsmesser)

E. JUNG and A. KRETSCHMER, *Betonstein-Zeitung* (Wiesbaden), V. 17, No. 6, June 1951, pp. 134-135

Reviewed by, RUDOLPH FISCHL

The Berlin-Dahlem material testing laboratory (Materialprüfungsanstalt) developed a handy and sturdy instrument, the "stress meter." It is used to measure the stress in prestressed steel wires by reading the deflection of the wire between two supports with a defined shear, which is immediately a measure for the prestressing. Each single wire can be checked and the scale is adjustable for different diameters of the wires. The instrument is easy to handle even by an unskilled person.

Reinforced concrete piles under tension

J. A. WAKEFIELD, *Engineering* (London), V. 172, No. 4471, Oct. 5, 1951, pp. 420-421

Reviewed by ARON L. MIRSKY

Tests of resistance to extraction of cast-in-place bored-type piles concreted under pneumatic pressure (rather than the weight of a mandrel). The large resistance to uplift developed by the piles enabled the designers to reduce the weight of concrete anchor blocks under the legs of transmission towers, saving considerable money. Methods of forming and testing the piles are described.

Steel-pan forms provide economical long-span roof

R. L. REID, *Civil Engineering*, V. 21, No. 9, Sept. 1951, pp. 32-33

Reviewed by ROBERT E. WILDE

Use of specially designed steel-pan forms for the reinforced concrete roof of a one-story brick building permitted the elimination of interior columns in various areas. The usual method of framing the roof proved practical and economical and contributed to speed of construction.

The pans were 3 ft square and were designed so that sides could be flexed for easy removal and the forms could be used on future jobs. Details of the concrete roof slab include 8 in. deep joists, 6 in. wide on the bottom, which were covered by a 2½-in. slab. The roof covered a total area of approximately 6000 sq ft divided into three panels. Each area was bounded by a system of concrete columns and beams, thus making it structurally a modified two-way slab.

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Research reports of the German Committee for Reinforced Concrete (Die Versuchsberichte des Deutschen Ausschusses fur Stahlbeton)

OTTO GRAF, German Committee for Reinforced Concrete, No. 105, 1951, Wilhelm Ernst and Son, Berlin. 46 pp.

Reviewed by ARON L. MIRSKY

A summary of the first 102 numbers (pre-1910 to 1950) of the Committee, prepared to acquaint German engineers with what had been accomplished, in particular view of the current dearth of research facilities in Germany and the wartime destruction of libraries. As an up-to-date summary of the present state of the art, as well as a condensed history of the past, it makes this reviewer wish for an American equivalent.

Approximate formulas for rectangular tanks (Naherungsformeln fur kastenformige Behalter)

H. CRAEMER, *Oesterreichische Bauzeitschrift* (Vienna), V. 6, No. 10, Oct. 1951, pp. 172-173

Reviewed by RUDOLPH FISCHL

The frequently used approximation to design the horizontal cuts of rectangular tanks as closed frames and the walls as cantilevers, is modified by using a new straight-line load distribution. By setting the maximum deflection of the frame equal to the deflection of the cantilever at a certain section, the load ratios are found, and the moments determined. This load distribution automatically takes care of the different conditions for high or for wide tanks. The formulas may be easily adjusted to different end conditions of the walls.

Composite action between precast reinforced concrete sections and cast-in-place concrete (Die Verbundwirkung zwischen Stahlbeton-fertigteilen und Ortbeton)

ADOLF MEYER, *Der Bauingenieur* (Berlin), V. 26, No. 11, Nov. 1951, pp. 327-330

Reviewed by ARON L. MIRSKY

Precast beams 3 m long were used as the tension portions of T-beams, with the compression flanges formed of concrete placed 60 or 120 days later. The beams were loaded at the third points in testing.

The author concludes that with static loading composite action existed without keyed joints or stirrup connectors embedded in the cast-in-place concrete to take the horizontal shear. However, excessive shrinkage of the cast-in-place concrete must be prevented.

It appears to this reviewer that more and better test data might be desirable; the data reported in this article appear open to some question, with the various experimental shortcomings mentioned by the author possibly being to blame.

Reports on experiments with continuous reinforcement in concrete pavements—California

THOMAS E. STANTON, *Proceedings*, Highway Research Board, V. 50, 1950, pp. 28-44

Presents data on the California continuous reinforced concrete pavement project designed to study stresses in the steel reinforcing and crack incidence in an 8-in. uniform thickness pavement slab with the steel centrally placed in the depth of the slab, as well as long-time performance with relation to riding qualities. Details are given of the installation of SR-4 strain gages and thermocouples in the slab and records of crack formation are reproduced.

Crack incidence was greater on sections placed early in the day than on adjoining sections placed late in the day. The project was constructed during summer of 1949 with consequent relatively high atmospheric temperatures. Up to one and one-half years the crack incidence was not materialy affected by the percentage of steel.

Effect of exposure to soils on the properties of asbestos-cement pipe

IRVING A. DENISON and MELVIN ROMANOFF, *Journal of Research, NBS*, V. 5, No. 4, Nov. 1951, pp. 367-379

Reviewed by C. C. FISHBURN

This report summarizes the results of study made on two varieties of asbestos-cement pipe involving the exposure of 15 different soils for periods up to 11 years. The soils range from well-aerated types deficient in water soluble salts to very poorly aerated ones containing high concentrations of soluble material. The effects of exposure to the soils on the mechanical and physical properties of asbestos-cement pipe are indicated by measurements of hydrostatic bursting pressure, crushing strength, water absorption, apparent specific gravity and by observations of the condition of the surface of the specimens removed. Bursting and crushing strengths of the pipe samples, after exposure, were without exception higher than the requirements of the Federal specifications for this class of material.

Reinforced concrete slurry tanks

OSCAR FABER, *Concrete and Constructional Engineering* (London), V. 46, No. 11, Nov. 1951, pp. 323-332
Reviewed by GLENN MURPHY

Ten mixing and storage tanks constructed for the British Portland Cement Manufacturers at Shoreham are described. The two pairs of preliminary mixing tanks are 54 ft 6 in. high and each has an outside diameter of 23 ft 7 in. In these tanks the chalk slurry and clay slurry are mixed with the assistance of air agitation. The two final mixing tanks are each 78 ft high with an outside diameter of 27 ft 1 in. and are built to form a pair. The four storage tanks are 66 ft in diameter and have an over-all height of 14 ft 8 in.

The walls and bottoms of the tanks vary in thickness, being proportioned in accordance with the hoop tension. In the final mixers the walls vary uniformly in thickness from 6 in. at the top to 16 in. at the bottom. The maximum design tensile stress in the concrete is 175 psi. Precautions were taken to avoid leakage, and as soon as possible after casting the tanks were filled with water to reduce shrinkage cracking, and to aid in strength gain.

Revolutionary staircase built of prestressed concrete

G. MAGNEL, *Civil Engineering*, V. 21, No. 9, Sept. 1951, pp. 25-27
Reviewed by ROBERT E. WILDE

Describes briefly the design of a helicoidal staircase built of prestressed concrete. The staircase, built for General Motors Continental, Antwerp, Belgium, rises 17 ft unsupported except at bottom and top.

Since no reliable formula exists for computing torsional stresses in very thin rectangular concrete sections, in this case about 11 in., Professor Magnel tested a full-scale model under loads up to 3½ times the working load. The final staircase design was based on the test results.

Prestressing of the model was by means of two cables, each containing 16 wires of 5 mm diameter, stressed at one end up to 145,000 psi.

Prestress was applied to the actual staircase by jacking both ends of six cables and was applied simultaneously to reduce the friction loss from 45 (measured in model) to 22.5 percent.

Construction of a large jetty on Southampton water

Concrete and Constructional Engineering (London), V. 46, No. 10, Oct. 1951, pp. 305-315
Reviewed by GLENN MURPHY

This article describes the unusual equipment used in the construction of the Esso Petroleum Co. jetty at Fawley, the design of which was described in the previous number of this journal. A casting yard with a capacity of 50 cu yd per day was established for casting the piles, frames, beams and struts. Both air-entraining and ordinary concrete were used. Since some of the piles weighed 18 tons, special handling equipment was necessary. Floating pile drivers were used. Caissons of 36- and 40-ft diameter were cast initially with a 24-ft wall. After seven days they were lowered to high-tide level and floated into a new position where the walls were extended to 66 ft. They were then moved to their final positions and sunk. A number of photographs taken at different stages of progress are included.

The ultimate resistance of prestressed concrete beams

P. W. ABELES, *Concrete and Constructional Engineering* (London), V. 46, No. 10, Oct. 1951, pp. 295-303
Reviewed by GLENN MURPHY

The author discusses the significance of one definition of factor of safety as applied to reinforced concrete beams and describes the differences in expected behavior of beams containing bonded reinforcement (pre-tensioned) and beams containing unbonded reinforcement (post-tensioned without effective grouting). The design of both types of beams is considered, and formulas and coefficients are given for the ultimate resisting moment expected in beams of rectangular cross section. It is assumed that the compressive stress in the concrete at failure is equal to 0.6 of the cube strength and is uniformly distributed throughout the area in compression. I-beams with bonded wires are also discussed.

As proof of the author's statement that the results from his formulas are conservative he cites the results from tests on three beams tested recently. The author states that extensive research is necessary to develop reliable design data and indicates that since only approximate results are now available the simplest solution seems to be preferable.

Coast protection works at Clacton

Concrete and Constructional Engineering (London), V. 46, No. 10, Oct. 1951, pp. 316-318

Reviewed by GLENN MURPHY

In an abstract of a paper by W. Aiston in the *Journal of the Institution of Municipal Engineers* the concrete groynes and sea wall at Clacton-on-Sea are described. The groynes are from 7 to 11 ft wide and from 5 to 11 ft high with stepped vertical sides and buttresses to reduce scour. The sea wall is 11 ft wide at the base and 15 ft 6 in. high with a battered seaward face. In addition, both faces of the wall are stepped. Construction details are described and photographs and drawings are included.

Durability of concrete with and without reinforcement subjected to weathering (La durabilite du beton et du beton arme soumis aux actions atmospheriques)

F. CAMPUS, *Cours de Constructions du Genie Civil*, Université de Liège, No. 79, Sept. 1950

Reviewed by PHILLIP L. MELVILLE

Poor durability has been observed on a number of reinforced concrete structures in Belgium and especially on poles for power lines. A theoretical study indicates the importance of keeping the voids to a minimum as reflected in the porosity which may best be determined by testing of the hardened concrete in the autoclave. It is advised that the concrete mix should be designed on the basis of absolute volumes with field quantities measured by weights. Both temperature and moisture variations are responsible for poor durability with freezing most critical. Frost will cause swelling of the concrete and shrinkage of the reinforcement with ensuing cracking. It is advisable to place a cover of sufficient thickness on bars near the corners. As soon as cracking has taken place, a progressive failure follows due to volumetric changes of the reinforcing steel upon corrosion. This action can also take place without cracking when all free lime has been leached out of a shallow covering of the steel. It is recommended that the porosity be less than 10 percent by volume and the cover on reinforcement be at least 2 diameters or 30 mm., whichever is larger. Suggestions are made for repairs of weathered concrete and specifications on thickness of cover in Norway, France, Finland, Great Britain and the United States are studied.

Thin-shell-arch hockey stadium for Quebec built in 10 months

ROBERT ZABOROWSKI, *Civil Engineering*, V. 21, No. 10, Oct. 1951, pp. 36-40

Reviewed by ROBERT E. WILDE

The thin-shell concrete arch, designed to house Quebec's (Canada) new hockey arena, spans 240 ft between A-frame buttresses. The actual construction was accomplished in about 6½ months, although this did not include all the interior finishing.

The curved roof slab, varying from 4 to 6 in. thick, is supported by arches 4 ft deep and 2 ft wide, spaced 30 ft on centers. The 4 x 2-ft arch-rib cross section is not a minimum limit from the point of view of stability. It was selected to permit rapid decentering of forms in cold weather and required only normal field control of concreting—both important economic considerations because of the short time allowed for completion.

ACI recommendations were followed as a general design code in designing the thin-shell-arch roof system.

The foundation design is a system of vertical and battered caisson-type piles capped by a slab. The Franki caisson pile, heretofore not used in the United States, was employed. It is produced by driving a casing to a bearing stratum. A dry concrete mix is forced into the steel shell; the shell is pulled as concrete is rammed down.

Ready-mixed concrete was used in the project. Approximately 6 sacks of cement, generally high early strength, were used per cu yd of roof concrete. Slumps did not exceed 3 in.; air entrainment was 4 to 6 percent. A membrane curing compound was used on the roof concrete.

New method of prestressing reinforced concrete bridges (Reconstruction du pont de Vaux-sur-Seine—Un nouveau procede de mise en tension des armatures dans un ouvrage precontraint)

L. MOINEAU, *La Technique Moderne-Construction* (Paris), V. 6, No. 9, Sept. 1951, pp. 301-307

Reviewed by ALEXANDER M. TURITZIN

A description of the prefabrication, erection and prestressing of a three-span continuous bridge. The middle span is 26 m long and each end span is 15 m long. The bridge is supported on two intermediate piers built on piles driven into the river bed and on two end piers located at the river banks. After the piers were in place, two steel trusses

15 m long were erected, thus forming two temporary end spans. Two adjacent concrete box girders of variable cross section and prefabricated in advance were slid in position over these trusses. The box girders were 28 m long with their cantilever ends meeting at the center of the river. Two layers of cables were placed inside the box girders and securely attached at their ends to specially designed hoop and spiral anchors. The cables were stressed by spreading them apart by means of jacks located in the plane of the cables. Thus the cables were elongated and tension induced, the intensity being regulated by the amount of spread.

Longitudinal tension wires were made of hard steel with an elastic limit of 110 kg per sq mm and an ultimate limit of 160 to 180 kg per sq mm. To avoid rusting, the cables were coated with a heavy grease applied hot. Aggregates for concrete were carefully graded. At 28 days the concrete showed an average strength of 460 kg per sq cm. It is believed that this inexpensive way of prestressing will find its application in the construction of many prestressed concrete structures.

Study of some special modern concrete mixes (Etude de quelques betons modernes de types speciaux)

MARIUS DURIEZ, *Revue des Materiaux de Construction* (Paris), No. 434, Nov. 1951, pp. 321-327; No. 435, Dec. 1951, pp. 356-60

Reviewed by PHILLIP L. MELVILLE

In recent years, two different trends have appeared in the design of concrete mixes. On the one hand concrete has been made less dense and on the other hand more compact, the latter being theoretically more sound.

Lightweight concretes of the first group are obtained by the addition of foaming or gas-producing agents. Density is between .8 and 1.2, they can be sawed, nailed, are fairly nonabsorbent, have rather good durability, excellent thermal and sound properties but low strengths.

In air-entraining concrete, around 4 percent is present in voids with diameters between 100 and 200 microns. Permeability is reduced. Air entrainment will lower the strength but due to the resulting increased plasticity, better workability, less segregation and more homogeneous distribution of voids (giving better durability) are obtained. Air-entrainment results simultaneously in reduced permeability and hygroscopicity of both mortar and concrete. In plastified or homogenized concrete, wetting agents are added to increase workability with possible reduction of the mixing water. Activated mortars and concrete can be prepared by special mixing resulting in a pseudo-colloidal dispersion similar to an emulsion. By using clean, gap-graded aggregates injected with activated mortar (such as "Colgrout"), "Colcrete" is obtained. The product is somewhat similar to cement macadam.

Excess water can also be removed by vibration or compaction. The use of vacuum can result in dense concrete by removal of water, especially in thin sections.

Influence of high temperatures on prestressed concrete (Invloed van hoge temperaturen op voorgespannen beton)

G. BAAR, *Communication* No. B 06, International Congress on Prestressed Concrete, Ghent, Belgium, Sept. 1951

Author reports his investigation of the fire resistance of prestressed concrete and presents data, derived from earlier tests and from more recent supplementary tests, on the variation of cube strength and modulus of elasticity of concrete with temperature. The effect of temperatures ranging from 32 to 1470 F on the tensile strength and yield point of high-strength steel is compared with the effect on the corresponding characteristics of mild steel.

A test is then described, made on a six months old beam 38 ft long and 25 in. deep, prestressed by means of a cable made up of twenty-four 0.2 in. diameter wires. This beam was placed in a fire-brick furnace heated by 22 gas injectors and was loaded with concrete blocks equivalent to the design load.

The maximum temperature reached in the lower flange was 1310 F; at the same time the temperature at the cable level ranged between 450 and 520 F, while the upper flange was at 212 F.

Large deflections, up to 7.6 in. occurred during the test. When the specimen was examined after the test, no deformations or displacements were observed of either the wires or the wedges. The high temperatures has not damaged the ends of the cables nor affected the quality of the wire steel.

From this investigation the author concludes that (a) a 2-in. concrete cover and

proper grouting provide adequate protection for the cables up to 1300-1475 F; (b) the ultimate compressive strength of high quality concrete appears to decrease considerably when exposed to high temperatures; (c) the behavior of a specimen during a fire-resistance test is not a wholly dependable base for predicting the performance of a member of very different size; (d) the additional deflection due to the fire-resistance test depends to a large extent on the intensity of the load which the specimen is carrying during the test; and (e) complete failure of prestressed concrete members is not to be expected in case of fire, but excessive deflections are more likely to occur than in ordinary reinforced concrete construction.

First conference on concrete and other applications of portland cement (Primera conferencia del hormigon y otras aplicaciones del cemento portland

Comite Permanente de las Conferencias del Hormigon (Buenos Aires), 1950, V. 1, 590 pp.; V. 2, 430 pp.
Reviewed by FRANCISCO J. CORDOVA

Two volumes representing the proceedings of the First Conference on Concrete and other Applications of Portland Cement were released in the latter part of 1950. The conference was held in Buenos Aires, Argentina, November 7 to 19, 1947, under the chairmanship of Juan Agustín Valle, noted Argentine technologist and Director of the Argentine Institute of Portland Cement. Other presiding members of the permanent committee of the Conference on Concrete were Enrique Gonella, Roberto Gorostiaga and Victorio Lavarello (Vice-Chairmen), Antonio Augusto Rotondaro (General Secretary), Carlos Luis Coqueugnoit (Treasurer), and José Gandolfo and Oreste Moretto (Secretaries).

The conference was attended by engineers from several countries, namely: Argentina, Brazil, Spain, Uruguay, Chile, France, Perú and the United States. Various papers were also submitted by engineers of other South American and European countries who were unable to attend. The reviewer of this interesting work was an attending member of the conference.

The American Concrete Institute was represented by Charles E. Wuerpel who read a paper, in Spanish, on air-entrainment. A paper on the same subject was submitted by F. R. McMillan.

A total of 77 papers dealing with concrete and cement comprise the two volumes.

There are 16 papers on technology, 10 on highways, 13 on hydraulic structures, 29 on structural considerations, 4 on architectural concrete, and 6 on rural construction.

The contents of these two volumes should be of great value to American engineers contemplating the expansion of their activities to South American countries. Many of the papers, principally those related to concrete mixture design, reinforced concrete, road construction and structural design are skillfully presented and well-illustrated by numerous graphs, diagrams and photographs. The papers on the theoretical phases and applications of reinforced concrete which the Argentines have highly developed should be of great interest to engineers and designers, specially those who show a preference for the mathematical analysis.

Long-time study of cement performance in concrete. Chapter 6—The heats of hydration of the cements

GEORGE J. VERBECK and CECIL W. FOSTER, ASTM
Proceedings, V. 50, 1950, pp. 1235-1262

The sixth report on the research project begun by the Portland Cement Assn. in 1940. The first four chapters of the "Long-time study" appeared in the ACI JOURNAL, February to June, 1948, and dealt with the history and scope of the project, manufacture of the test cements, chemical and physical tests of the cements, and microscopic study of the clinker. The fifth chapter, "Concrete Exposed to Sulfate Soils," was a special publication of ACI and PCA. Chapter 7, reporting on the performance of the test cements in the New York test road, appeared in the ACI JOURNAL, June, 1951.

Based on the data presented in Chapter 6, the authors conclude that for neat cement pastes having a water-cement ratio of 0.40 by weight that at this water-cement ratio:

1. The heat of hydration of portland cement increases continuously for at least 6½ years.

2. The composition of the cement affects the heat of hydration at all ages. The heats of hydration of types II, IV and V cements were consistently below those of the types I and III cements at all ages up to 6½ years. Between 1 and 6½ years the rate of heat liberation of the types II, IV and V cements was less than for types I and III cements.

3. The use of flake Vinsol resin as an air-entraining agent had no significant effect on the heat of hydration.

4. The results of the least squares analyses indicate that somewhat better correlation between the heats of hydration and the calculated phase compositions of the cements than between the heats of hydration and the microscopically observed phase compositions.

5. Temperature of curing appreciably affects the rate of heat liberation at early ages. However, this effect does not persist, as companion pastes cured at 70 and 100 F had approximately the heats of hydration at 28 days.

6. The heat of hydration of cements at ages beyond 28 days cannot be estimated from strength data as other factors are involved in the relationship.

7. General relationships were observed between heat of hydration and either the nonevaporable water content or the vapor sorption of the pastes, although cement composition exerts some influence on these relationships.

Tests on concrete, with electrical-resistance strain gauges

N. S. J. Grassam and David Fisher, *Engineering* (London), V. 172, No. 4469, Sept. 21, 1951, pp. 356-358
Reviewed by Aron L. Mirsky

Describes methods of applying SR-4 gages to both damp and dry concrete surfaces, and results of tests on plain concrete under direct loading (tension and compression), using SR-4 gages and mechanical gages for comparison.

The method of affixing the gages to dry concrete surfaces is similar to Anderson's (*Engineering News-Record*, March 8, 1951, pp. 46-47; ACI Journal, *News Letter*, June 1951, pp. 6-9).

1-in. gages were found to yield good results on concrete made with $\frac{3}{8}$-in. aggregate.

Compression tests on 10 x 5-in. diameter cylinders indicated that the stress-strain curves closely approximated a parabola with the vertex at the ultimate; the curves were not continued beyond the ultimate ("fracture"). The ratio of lateral to longitudinal strains (Poisson's ratio) varied with the loading; the authors point out that length of the specimen may have been sufficient to make the barrelling due to end constraint (due to the platens) negligible. It should be noted that a soft rubber sheet $\frac{1}{16}$ in. thick was placed between the cylinder ends (caps) and the platens to assist in obtaining a uniform pressure distribution, since the platens were not ball-seated.

In the tension tests, the arrangement of gages on the specimens was purposely made such as to indicate any eccentricity of loading; the results were quantitative.

In the October 12 issue of *Engineering* (p. 468), K. R. Peattie describes a similar investigation using a 9 x 4½ in. diameter cylinder; the SR-4 gage lengths were ½ in., four being mounted on each of two diametrically-opposed generators of the cylinder; rubber sheets were not used. Strain readings increased from either end of the cylinder towards the midheight; the writer questions whether the soft rubber sheets used by the authors reduced the effect of end constraint and thereby the variation of strain along the height of the cylinder.

In the issue of October 26 (p. 531), the authors take exception to this, stating that such barrelling is theoretically very small, although a certain amount was found with ⅛ in. mill-board used between cylinders and platens.

The effect of lateral constraint at the ends of the test cylinders, due to friction between cylinders and platens, is further discussed by Peattie in the Nov. 9 issue (p. 596) and by the authors in the Dec. 21 issue (pp. 788-789).

Prestressed concrete (Beton precontraint)

Y. Guyon (Preface by E. Freyssinet), *Institut Technique du Batiment et des Travaux Publics* (Paris), 728 pp. 4704 fr. (about $15.00)
Reviewed by Paul Rogers

This truly remarkable book of 728 pages with 503 figures could really be called the "Bible" of prestressed concrete. It contains just about everything known at this time about this new structural system.

The book is divided into three major sections. The first section deals with the basic fundamentals and the different systems are studied both theoretically and experimentally. The second section presents the precise elastic calculations with special emphasis on problems which occur most frequently. Many of these problems are solved numerically. The third section is devoted to the general review of the problems, the subject of the factor of safety is analyzed

and new methods based on a combination of elastic and plastic theories are proposed.

More specifically the first section includes simple examples of prestressed beams, beams composed of blocks, elementary stress distribution in prestressed beams, simple examples of jacks and cable anchorages, practical studies of the Marne bridge and the Cherbourg sea-wall caissons, loss of prestress due to long term loading, instantenuous and retarded deformations, plastic flow of concrete as a factor in reducing or eliminating prestress in the cables, recommendation for the use of high tensile strength wires and cables, suggestion for allowing for the relaxation of the steel stress in establishing the initial prestress, effect of live loads on original stresses, fundamental differences between reinforced concrete and prestressed concrete, advantages of curved cables, limitations of live load compensation, prestressing by the use of expansive concrete, effect of shear stresses, the Freyssinet anchorage cone, different systems of prefabrication, the injection of grout, tools and machinery, the Blaton-Aubert system of anchorage, prestressing of the Montesson tunnel (slippage of the anchorages), prestressing without steel, materials, residual deformations, deformations due to repeated loadings, shrinkage and swelling, compressive and tensile strength, different wires and cables, deformation and relaxation of steel, influence of cable friction, fire resistance, codes and specifications of fire resistance, stresses on the abutments due to prestressing, stress concentrations, influence of axial loads, hoop anchorages, pretensioned (bonded) wires, and stresses in abutments of pretensioned systems.

The second section is devoted to the elastic theory as applied to prestressed members. Discussed are beams of constant moment of inertia and uniform cable cross section, exterior and interior forces, proof of stability, initial and permanent prestress, evaluation of the drop in the tension cables, the value of m, moduli of elasticity, neutral axis, practical computation of sections, compensation of live loads by prestressing, rectangular slabs and beams, reversed stresses, pattern for cables, resistance to shearing forces, inclined cables to reduce the effect of shear stresses, empirical rules for safety stirrups, formulas, beams of constant cross section with pretensioned prarallel wires, balanced design of pretensioned wires to resist a predetermined moment, beams of constant cross section with raised cables, successive elevation of several cables, shape of different elevated cables, beams of variable moment of inertia, the case of totally compensated loads, beams similar to tied arches, resistance to shearing stresses, shearing stresses caused by prestressing, distribution of shearing stresses and minimum secondary reinforcements.

The third section deals with the problems of safety, plasticity and a general review of all the studies made. Included are investigation of a 25 ft long parabolic beam with the maximum depth at the center, description of a continuous hinged (Gerber-type) bridge in Toulouse, principal and transverse deformations, report on the breaking of some isolated joists with pretensioned wires, flexural studies, studies on shear and bond, safety factors with regard to concrete crushing, flexure and shear, report on some systems of joist prefabrication, static and dynamic investigations concerning bond, studies on cracks of rectangular beams, influence of mild steel as longitudinal reinforcement, proposed empirical rules for rectangular beams, resume of studies on prestressed concrete beams, deformations and crackings, under-reinforced beams, limitations of law or breaking moments, influence of the steel tension on breaking moments, failure without previous cracking, fragile pieces, report on the shear stresses of the Djedeida bridge, factors of safety, elasto-plastic analysis, verification of the safety factors based on limit design, justification of safety factors in prestressed beams, practical examples, tables and charts, direction of shear cracks, and laws of shear stress variations for beams of variable moment of inertia.

Although this book is very extensive and complete in itself, this reviewer believes that the complete omission of all references to non-French works on prestressed concrete tends to detract from the value of the book. It is believed that the great advancements, both theoretical and practical, developed in English- and German-speaking countries should have been given recognition in a book of such extent.